UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION II
245 PEACHTREE CENTER AVENUE NE, SUITE 1200
ATLANTA, GEORGIA 30303-1257

May 12, 2011

Mr. Jon A. Franke
Vice President, Crystal River Nuclear Plant
Crystal River Nuclear Plant (NA1B)
15760 West Power Line Street
Crystal River, FL 34428-6708

SUBJECT: CRYSTAL RIVER NUCLEAR PLANT – STEAM GENERATOR REPLACEMENT INSPECTION PROGRESS REPORT 05000302/2011009

Dear Mr. Franke:

On April 22, 2011, the U.S. Nuclear Regulatory Commission (NRC) completed inspections at your Crystal River Unit 3 Nuclear Plant in accordance with NRC Inspection Procedure (IP) 50001, Steam Generator Replacement Inspection. The enclosed inspection report documents inspection results, which were discussed on April 28, 2011, with you and members of your staff.

The inspections examined activities conducted under your license as they relate to safety and compliance with the Commission's rules and regulations and with the conditions of your license. The inspectors reviewed selected procedures and records, observed activities, interviewed personnel, and conducted plant walk downs, including visual examination of accessible portions of the containment building structure.

In accordance with 10 CFR 2.390 of the NRC's "Rules of Practice," a copy of this letter and its enclosure will be available electronically for public inspection in the NRC Public Document Room or from the Publicly Available Records (PARS) component of the NRC's Agencywide Documents Access and Management System (ADAMS). ADAMS is accessible from the NRC Web site at http://www.nrc.gov/reading-rm/adams.html (the Public Electronic Reading Room).

Sincerely,

/RA/

Mark E. Franke, Chief
Operations Branch 2
Division of Reactor Safety

Docket No. 50-302
License No. DPR-72

Enclosures:
1: Inspection Report 05000302/2011009
w/Attachment: Supplemental Information
2: Licensee Question Tracking Database

cc w/encl: (See page 2)

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X PUBLICLY AVAILABLE ☐ NON-PUBLICLY AVAILABLE ☐ SENSITIVE X NON-SENSITIVE

ADAMS: ☐ Yes ACCESSION NUMBER: ML__________ ☐ SUNSI REVIEW COMPLETE

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Letter to Jon A. Franke from Mark E. Franke dated May 12, 2011.

SUBJECT: CRYSTAL RIVER NUCLEAR PLANT – STEAM GENERATOR REPLACEMENT INSPECTION PROGRESS REPORT 05000302/2011009

Distribution w/encl:
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RidsNrrPMCrystal River Resource

U.S. NUCLEAR REGULATORY COMMISSION

REGION II

Docket No.:	50-302
License No.:	DPR-72
Report No.:	05000302/2011009
Licensee:	Progress Energy (Florida Power Corporation)
Facility:	Crystal River Unit 3
Location:	Crystal River, FL
Dates:	January 1, 2011 through April 22, 2011
Inspectors:	R. Carrion, Senior Reactor Inspector L. Lake, Senior Reactor Inspector
Approved by:	Mark E. Franke, Chief, Operations Branch 2 Division of Reactor Safety

SUMMARY OF FINDINGS

IR 05000302/2011009; 01/01-04/22/2011; Crystal River Unit 3; Steam Generator Replacement Inspection

This report covered an infrequently performed Steam Generator Replacement Project (SGRP) inspection performed by regional reactor inspectors from January 1, 2011, through April 22, 2011. This report also includes a list of issued inspection reports and a summary of the SGRP inspections performed prior to December 31, 2010. The NRC's program for overseeing the safe operation of commercial nuclear power reactors is described in NUREG-1649, "Reactor Oversight Process," Revision 4, dated December 2006.

A. NRC-Identified & Self-Revealing Findings

No findings were identified.

B. Licensee-Identified Violations

None.

REPORT DETAILS

Background

The licensee scheduled the replacement of its steam generators during Crystal River Unit 3 (CR3) Refueling Outage 16, which began in September 2009. The steam generators are housed in the reactor containment building. In preparation for the steam generator replacement, the licensee evaluated options for moving the existing steam generators out of the containment building and moving the new steam generators into the containment building. The licensee decided to make a construction opening in the containment building wall, approximately forty feet directly above the equipment hatch, to facilitate this evolution. Fabricating the opening in the containment building wall included preparing the containment building by detensioning its pre-stressed tendons and then removing the concrete with high pressure water (a process known as hydro-demolition), removing the rebar and tendons from the area of the construction opening, and cutting the containment liner plate. A concrete delamination in Bay 3-4 of the containment structure was discovered while creating the SGR opening in October 2009. The licensee determined that the root cause of the delamination was related to the scope and sequence of the tendon detensioning process.

The licensee's containment building repair plan included: (1) additional detensioning of containment; (2) removal of delaminated concrete; (3) installation of reinforcement, including radial reinforcement through the delamination plane; (4) placing of new concrete; (5) retensioning containment; and (6) post-repair confirmatory system pressure testing. In early 2011, the licensee had completed repair steps 1 through 4 and was in the process of retensioning the containment. On March 14, 2011, during the final stages of the re-tensioning process, the licensee had indications that a new delamination occurred in Bay 5-6 of the containment structure.

Since September 2009, and through the end of this inspection period, Crystal River Unit 3 has remained shutdown, a mode of operation where containment building operability is not required.

The purpose of this inspection report is to document all inspection activities performed related to the steam generator replacement project for Unit 3 containment restoration activities including repairs due to the concrete delamination identified in October 2009.

4. OTHER ACTIVITIES [OA]

4OA5 Steam Generator Replacement Inspection (IP50001)

.1 SGRP Inspection Activities through December 31, 2010

Inspection of the licensee's SGRP began in September 2009 when the inspectors reviewed the preparations for heavy load movement and lifting and started the review of the design modifications associated with the project. Inspection of the licensee's SGRP continued in the fourth quarter of 2009 and throughout 2010. Results of the SGRP inspections are documented in the quarterly integrated resident inspector reports. The following is a summary of the activities inspected. Details of the inspections can be found in the following inspection reports:

Inspection Report	ADAMS ML #	Report Section
05000302/2009004	ML093030165	4OA5.2
05000302/2009005	ML100250014	4OA5.2
05000302/2010002	ML101170619	4OA5.3
05000302/2010003	ML102090239	4OA5.2
05000302/2010004	ML103020127	4OA5.3
05000302/2010005	ML110270190	4OA5.2

Design and Planning

The inspectors reviewed and examined the SGRP activities and compared them to the requirements of the American Society of Mechanical Engineers (ASME) Code. The inspectors reviewed Engineering Change (EC) 63038, Replacement Once Through Steam Generators (ROTSGs or RSGs), which included the design changes, analyses, evaluations, safety analyses, 10 CFR Part 50.59 change evaluation, configuration, materials, implementation, and post-modification testing acceptance. The inspectors reviewed EC 62500, RCS Hot Leg Cutting and Welding, EC 63016, Containment Opening; EC 63025, Main Feedwater Flow Accelerated Corrosion (FAC) Pipe Replacement, EC 63026, RCS Cold Leg Cutting and Welding, EC 63027, Secondary Side Large Bore Pipe Cutting and Welding, EC 63034, Structural Interferences, and EC 63039, Replacement Steam Generator Anchorage. The inspectors also reviewed selected work order (WO) packages prepared for the construction and implementation of the ECs to determine whether appropriate work processes and quality control hold points were implemented.

Steam Generator Removal and Replacement

During the hydro-demolition process to create the construction opening in the containment wall, the licensee identified concrete cracks/separations. The concrete separations were located within the entire perimeter of the opening. An NRC Special Inspection Team was chartered to inspect the separation issues. Results of the Special Inspection, including the licensee's root cause analysis, are documented in NRC Inspection Report 05000302/2009007 (ML1028610261). The licensee evaluated the containment wall cracks to modify the horizontal transfer system (HTS) supporting structures. Prior to the removal of the original steam generators (OSGs), the inspectors reviewed, observed, and evaluated the associated temporary and permanent modifications of the cutting, disconnecting, and the providing of temporary supports for the OSGs and cutoff piping. The inspectors observed lifting, rigging, downending and upending, and transporting of the OSGs, RSGs, and associated equipment; machining and preparations of the existing piping for the connections to the RSGs; welding and non-destructive examination (NDE) activities; and the radiological safety plan for the temporary storage and disposal building of the retired steam generators. The inspectors reviewed and observed the major structural modifications. The inspectors observed the licensee performance inspection of the steam generator hold-down bolts to verify that the bolts were acceptable to hold down the RSGs after the OSGs were moved from their cubicles. During the steam generator (SG) removal and replacement, the inspectors observed licensee activities associated with controls for excluding foreign material, including the primary and secondary side of the steam generators and in the related RCS openings, and the establishment of operating conditions including defueling, RCS draindown and system isolation. The inspectors also reviewed procedures, examination results, modification packages, and WO packages related to the modifications, including the construction opening steel containment vessel (SCV) reinstallation, to ensure compliance with the requirements of the ASME Code.

RSG Fabrication, Preservice Inspection, and Baseline Inspection

The inspectors reviewed records associated with the materials, fabrication, examination, and testing for the RSGs, and replacement hot leg piping subassemblies ("Candy Canes"), to verify compliance with the ASME Code. The inspectors also reviewed documentation and interviewed plant personnel regarding the pre-service and baseline testing of RSG tubing. The inspectors also reviewed documentation regarding the manufacture of the RSG tubing, including heat treatment records and nonconformance reports.

Welding

The inspectors reviewed a sample of welding activities associated with the installation of the RSGs to evaluate compliance with licensee/contractor procedures and the applicable ASME Code. The inspectors reviewed joint configuration drawings, welding procedures, welding specifications, welding procedure qualifications, welder qualification records, weld data records, nuclear condition reports (NCRs), and post-weld heat treatment procedures.

Non-Destructive Examination

The inspectors reviewed the NDE procedures, calibration and examination reports, and NCRs, and observed in-process NDEs, including liquid penetrant examinations (PTs), magnetic particle examinations (MTs), radiographic examinations (RTs), and ultrasonic examinations (UTs), and compared them to the requirements of the procedures and the ASME Code for the construction, pre-service, and baseline inspections.

Containment Construction Opening and Closure - Steel and Concrete Containment

The inspectors reviewed the licensee's activities associated with the concrete removal and the removal and restoration of the steel containment liner plate (SCLP) for the containment construction hatch opening, as detailed in the EC 63016, Containment Opening. The inspectors reviewed the plans for the cutting and restoration of the SCLP for the construction opening and compared post-testing requirements to the applicable ASME Code. The inspectors observed the hydro-demolition of concrete for the containment construction opening and reviewed the WO packages for the cutting of the liner plate to verify that the steps had been completed and documented. The inspectors also reviewed the welding procedures, procedure qualification records, and welder qualification records to confirm that the Code-required essential and supplemental essential welding variables were met. The inspectors reviewed the WO packages, including welding electrode receipt inspection, vacuum box leak testing, MT records, material certification records, and qualification and certification records for NDE personnel, equipment, and consumables.

Heavy Load, Rigging, Lifting, and Transporting Activities

The inspectors reviewed the SG lifting preparation activities and lifting equipment load test data to ensure that they were prepared in accordance with regulatory requirements, appropriate industrial codes and standards, and to verify that the maximum anticipated loads to be lifted would not exceed the capacity of the lifting equipment and supporting structures. The inspectors reviewed procedures, calculations, drawings, work packages, crane and equipment operator training and certificates, and load and function test records to verify that they were in accordance with regulatory requirements and appropriate industrial codes and standards. The inspectors also examined SGRP lifting, rigging, and transporting equipment, including the polar crane, mobile crane, the Temporary Lifting Device (TLD), the Horizontal Transfer System (HTS) (including its skid system), the down/upender device, the Outside Lift System (OLS), and the self-propelled modular transporter (SPMT). The inspectors observed a selective sampling of rigging, lifting, transportation, and positioning of the original and replacement SGs.

Quality Assurance (QA) Program and Corrective Actions

The inspectors conducted a review of the quality assurance program and its implementation for the SG replacement to assess compliance with the requirements of 10 CFR Part 50, Appendix B. The inspectors also reviewed the surveillance reports and nonconformance reports issued for the root cause analyses, evaluations, repairs, or disposition during the manufacturing of the RSGs.

SG Post-Installation Verification and Testing

The inspectors reviewed the SG post-installation verification and testing program to verify that the required post-installation verification and testing, procedural changes, and the adjustment of the instruments were properly identified.

Containment Detensioning

The inspectors conducted a review of the licensee's detensioning activities for the repair of the delaminated containment wall and restoration of the containment wall to it pre-construction opening condition, including associated ECs and Work Packages (WPs). The inspectors also observed vertical and horizontal tendon detensioning. The inspectors observed and reviewed the records of the acoustic monitors and strain gauges used to detect sound volumes and concrete strain changes potentially due to new cracks or compressive or tensile stress changes in the concrete during the detensioning process. The inspectors reviewed the procedures, drawings, calibrations, equipment and personnel qualifications, and the tendon detensioning communication plan associated with detensioning to verify that the licensee performed the activities in accordance with approved procedures.

Concrete Removal, Surface Preparation, and Concrete Placement Activities

The inspectors conducted a review of the licensee's activities associated with the removal of the damaged concrete and restoration of the containment wall. The inspectors reviewed associated documents, including ECs, WPs, specifications, drawings, test reports, and NCRs. The inspectors observed the process of the hydro-demolition of damaged concrete, the surface preparation of concrete after the hydro-demolition, and pull-out testing to assure that the concrete surface would have enough tensile strength to bond the new and original concrete. The inspectors reviewed radial rebar drilling; grouting; and identified void problems, and their respective resolutions. The inspectors observed rebar and formwork installation and tendon sleeve condition in preparation of the concrete pour. The inspectors also reviewed the associated engineering packages, WPs, and drawings to verify that licensee activities were performed in accordance with approved documents. The inspectors observed concrete placement activities to verify that activities pertaining to concrete delivery time, flow distance, layer thickness, etc. conformed to industry standards established by the American Concrete Institute (ACI). The inspectors also observed that concrete placement activities were monitored by the licensee and contractor's quality control personnel and engineers. The inspectors observed in-process concrete testing and reviewed the results for slump, air content, temperature, and unit weight, to verify that this was done in accordance with applicable American Society for Testing and Materials (ASTM) requirements. The inspectors checked the batch plant for its certification and reviewed its preparation for the concrete pour. During the concrete placement activities, the inspectors identified a finding of very low safety significance and an associated non-cited violation (NCV) of 10 CFR Part 50, Appendix B, Criterion IX, "Control of Special Processes," for the licensee's failure to establish measures to assure that testing of rebar splices would adhere to the requirements of ASME Boiler and Pressure Vessel Code. (Refer to Section 4OA5.3 of Inspection Report 05000302/2010004 (ML103020127) for additional details about NCV 05000302/2010004-03, Failure to Submit Production Splices of Swaged Mechanical Splices for Testing.)

Containment Dome Cracks

On April 14, 1976, a delamination was identified in the containment dome during the final stages of containment construction and before initial plant startup. The area of the delaminated concrete was approximately circular in shape with a 105-foot diameter. The dome repair process included removal of the delaminated dome cap; removal of meridional, hoop, and radial reinforcement; and placement of a new dome cap. Instrumentation was installed to monitor the dome during tendon detensioning, retensioning and the initial structural integrity. As part of the Special Inspection to assess the circumstances associated with the delamination discovered in 2009, NRC inspectors conducted a review of the licensee's conclusion that the delamination identified in 1976 is not related to the current delamination. Additional background information and the results of these inspections are presented in NRC Inspection Report 05000302/2009007 (ML1028610261).

In 2009, NRC Special Inspection Team inspectors conducted concrete surface inspections on the containment dome and identified a rough and uneven surface condition of the dome surface. The inspectors reviewed the evaluation and corrective action that determined the rough and uneven surface condition of the dome has existed since the 1976 repair. The licensee had periodically completed numerous surface patches in an attempt to address the surface spalls. Following additional reviews of the dome tendon stresses and monitoring, the uneven surface also appeared to be a result of concrete installation and finishing from the 1976 repair and not related to settlement of the dome or the 2009 containment concrete wall delamination issue.

In 2010, the inspectors reviewed a condition assessment of the containment dome documented in the licensee's engineering change package EC 74801. The licensee had included this assessment in the activities associated with its 2009 containment extent-of-condition investigation. Included was Impulse Response (IR) testing and core bores made in support of evaluating the IR data. Anomalies were identified and, to evaluate the anomalies, additional examinations were performed. A total of about 10,000 points were tested and a total of 30 core samples were removed. Visual inspections were performed of each core sample and a video scope inspection of each core hole was performed after the core sample was removed. This evaluation revealed cracking in the plane of the dome (laminar cracking). The licensee determined that these anomalies were remnants of the repairs performed in 1976.

The inspectors reviewed the CTLGroup Project No. 059176 – Dome Report, which included the results of the examinations identified above. The information contained in this report was subsequently utilized in an engineering evaluation documented in Containment Dome Evaluation, Report No. CR-3-LI-537934-52-SE-0059. The engineering evaluation determined that the repairs made to the dome structure in 1976 are intact; that there are no significant anomalies, discrepancies, or structural issues which would affect the overall structural integrity of the dome structure; and that the structure is capable of performing its design basis functions as described in the Updated Final Safety Analysis Report (UFSAR).

Additional information is presented in NRC Inspection Report 05000302/2009007 (ML1028610261).

Containment Tendon Retensioning Plan

The inspectors reviewed the containment tendon retensioning plan, testing plan, and schedule. The inspectors also interviewed licensee personnel and reviewed documents related to the retensioning and testing plans. The licensee conducted a detailed analysis to develop a tendon retensioning sequence that would minimize the possibility of causing new cracks or delaminations in the containment during the retensioning process. The retensioning process began in January 2011. The inspectors also reviewed licensee plans for Containment Building testing after completion of tendon retensioning and post-maintenance testing after restart.

.2 SGRP Inspection Activities January 1, 2011 through April 22, 2011

Discussion of Technical Issues

The following issues were discussed with licensee personnel during this inspection period:

Bulges of Liner Plate

The inspectors completed a review of the licensee's actions related to containment liner bulges. The licensee developed a calculation to evaluate bulges in the CR3 containment liner plate. It was directed at determining an apparent cause for the bulges and establishing an analytically-based acceptance criterion for the bulges within the CR3 design basis. The analyses included finite element modeling of the liner and the associated anchorage to the concrete containment structure. The apparent cause for the bulges was determined to be a combination of elements, including geometrical imperfections in the original liner plate during construction. The calculations considered worst case configurations and a threshold for bulge size was established considering the effects that occur due to normal operation and accident conditions. The primary variables in the bulge evaluation were determined to be bulge size and thermal loading. The calculation found that the bulges have an insignificant effect on the response of the structure due to various load combinations. The current bulges are bounded by the acceptance criteria in the analysis. To ensure that conditions are acceptable in the future, the licensee planned to include the bulges in the IWE program. The licensee added a summary evaluation to the EC, which includes steps to validate the effect of retensioning on bulge size by measurement and evaluation of a representative sample before initiating Structural Integrity Test (SIT) pressurization as well as requirements to perform a complete baseline scan after completion of the SIT.

50.59 Evaluation

The inspectors reviewed the licensee’s evaluation of the containment building modification resulting from the introduction of the construction opening and its subsequent restoration with respect to requirements of 10 CFR, § 50.59, Changes, Tests and Experiments, to verify that the design bases, licensing bases, and performance capability of the containment had not been degraded through the modification and to verify that the design and license basis documentation used to support changes reflect the design and license basis of the facility after the change had been made.

The inspectors’ review remained ongoing at the end of the inspection period. Remaining activities necessary to complete the 50.59 review included: verification that tendon retensioning activities and containment testing validated licensee design assumptions; verifying that post-modification testing adequately confirmed containment functionality via the scheduled Structural Integrity Test (SIT) and Integrated Leak Rate Test (ILRT) prior to unit startup; verifying that design basis documentation used to support changes and design basis documentation affected by changes had been adequately updated and reflected the modified design and license basis of the facility consistent with the restoration; and verifying that the license’s UFSAR had been updated accordingly.

Vertical Cracks of Containment Building

One of the licensee’s design assumptions for containment repair was that vertical cracks discovered on the exterior wall of the containment building would close as the building’s tendons were retensioned. The inspectors walked down selected vertical cracks being monitored by the licensee to evaluate their condition. The licensee had measured the cracks periodically and determined that they were closing as the tendon retensioning process continued. The inspectors also visited the tendon control center where the retensioning process was controlled, and which housed the acoustic monitoring and strain gage instrumentation, and interviewed personnel in the center to better understand the operation of the systems being used and how the information obtained was interpreted.

The inspectors’ review of vertical cracks remained ongoing pending inspection of the containment building after all repairs are completed and tendons are fully retensioned, and the completion of the SIT and ILRT.

Tendon Re-tensioning Activities

The inspectors reviewed the licensee’s re-tensioning plans, procedures, and drawings. In addition the inspectors observed some of the re-tensioning work being performed on selected hoop tendons to verify that the work was being conducted per approved procedures.

SIT/ILRT Preparations

The inspectors interviewed licensee personnel responsible for the planned SIT/ILRT to determine the status of the test preparations; walked down the containment building to verify the locations of the extensometers to be used to measure the containment movements during the SIT/ILRT; and discussed the licensee's procedures to ensure that they conformed to industry standards and ASME Code requirements.

Events of March 14, 2011

On the afternoon of March 14, 2011, the licensee had completed the first retensioning sequence (Sequence #100, Hoop Tendons 42H41, 62H41, and 64H41) of the final pass (Pass #11). Per procedure, the licensee was waiting for the containment building to stabilize before beginning the next sequence and monitoring the structural behavior of the containment building via acoustical emissions monitors and strain gauges, specifically placed at various points of the structure to detect any abnormal/unexpected response to tendon retensioning. During this monitoring period, the strain gauges indicated an increase in strain and then failed high, and the acoustic monitors indicated a high level of acoustic activity in the bay bordered by Butresses #5 and #6 (Bay 5-6). The phenomenon reportedly lasted for about twenty minutes. The licensee conducted impulse response (IR) non-destructive examination NDE techniques to determine the condition of the wall in Bay 5-6. The IR scans of the bay determined that there were numerous indications consistent with a delamination. By the end of the inspection period, the licensee had determined that the delamination was extensive in Bay 5-6 and was continuing to evaluate the condition of the entire containment structure. Future inspection activities by the NRC relating to the March 14, 2011, event are to be determined.

4OA6 Meetings, Including Exit

Exit Meeting Summary

On April 28, 2011, the inspectors presented the inspection results to Mr. J. Franke, Site Vice President, and other members of licensee management via a telephone call. The inspectors confirmed that proprietary information was not provided or examined during the inspection.

ATTACHMENT: SUPPLEMENTAL INFORMATION

KEY POINTS OF CONTACT

Licensee personnel:
S. Cahill, Manager, Engineering
P. Dixon, Progress Energy
P. Fagan, RNP Technical Services Superintendent
G. Flavors, Nuclear Upgrades
J. Franke, Site Vice-President
T. Howard, Engineering
J. Holt, Site General Manager
J. Huegel, Nuclear Oversight
R. Knott, NPC Lead Engineer
M. Rigsby, Manager – Support Services

NRC personnel:
D. Rich, Chief, Branch 3, Division of Reactor Projects
T. Morrissey, Senior Resident Inspector
R. Reyes, Resident Inspector

LIST OF ITEMS OPENED, CLOSED AND DISCUSSED

Opened
None.

Closed
None.

Discussed

05000302/2010004-03	NCV	Failure to Submit Production Splices of Swaged Mechanical Splices for Testing

LIST OF DOCUMENTS REVIEWED

SGRP Activities January 1, 2011 through April 22. 2011

Procedures

Progress Energy

PT-178T, Special Procedure - Reactor Building Concrete Structural Integrity Test, Revision 0

SP-178T, Containment Leakage Test -Type "A" Including Liner Plate, Revision 0

Mistras Group, Inc.

Crystal River Unit 3 Tendon Retensioning Monitoring Procedure, Revision 4

Precision Surveillance Corporation (PSC)

Field and Quality Control Procedures

3.0 Receiving, Handling and Storage, Revision 0
3.1, Equipment Proof Test, Revision 0
5.0, Tendon Initial Degreasingand Cap Removal, Revision 1
6.0, Tendon Detensioning/Removal for Possible Reuse, Revision 0
8.0, Plasma Cutting Tendon Detensioning, Revision 0
8.1, Ram Tendon Detensioning, Revision 2
9.0, Monitor Tendon Force (Lift-Offs), Revision 1
10.0, Tendon Removal, Revision 0
11.0, Tendon Void Cleaning, Revision 0
13.0, Tendon Installation, Revision 2
14.0, Tendon Field Anchor Head and Buttonheading Application, Revision 2
15.0, Tendon Restressing, Revision 3
15.1, Anchorage Inspection of Stressed Tendon, Revision 1
15.2, Bearing Plate Concrete Inspection, Revision 0
15.5, Additional Vertical Tendon Restressing, Revision 0
16.0, Grease Cap Replacement, Revision 0
17.0, Grease Replacement, Revision 1

Quality Assurance Procedures

10.0, Calibration of Measuring and Test Equipment, Revision 3
10.1, Verification of Calibrated Status of Hydraulic Pressure Gauges, Revision 0

Nuclear Condition Reports

NCR	Date Issued	Description/Title
378555	01/29/10	DBD 1/1, Containment, Has an Incorrect Value for Tendon Wire
422131	01/13/11	This NCR Tracks SGT NCR 154 Liner Plate Coatings Damage
422383	01/14/11	This NCR Tracks SGT IIRP 144 Dropped Hard Hat
422487	01/15/11	This NCR Tracks SGT IIRP 145 Finger Injury
422488	01/15/11	This NCR Tracks SGT NCR 155 Tendon Buttonhead Discrepancies
440743	01/05/11	This NCR Is to track SGT NCR 148 Broken Tendon Wires
440778	01/05/11	Lost 0-2 Inch Dial Indicator
440785	01/05/11	This NCR TrackS SGT NCR 149/IIRP 135 Overlapping Shims
440833	01/05/11	Transposition Error In Attachment Z50 Caused Work Stoppage
441233	01/07/11	This NCR Tracks SGT NCR 150 Bent Test Wire
441239	01/07/11	This NCR Tracks SGT NCR 151 Liner Plate Coatings Defects

441332	01/07/11	This NCR Is to Track SGT IIRP 136 Dropped Object
441366	01/08/11	This NCR Tracks SGT NCR 152 Shim Stack Inconsistent
441394	01/09/11	SGT IIRP 138 Dropped Pendent Controller
441396	01/09/11	Bent Wire On Tendon 13H18 at Butress 3
441453	01/10/11	This NCR Tracks SGT IIRP 140 Adverse Trend
441648	01/11/11	This NCR Tracks SGT IIRP 141 Dropped Object
442571	01/17/11	This NCR Tracks SGT NCR 156 Tendon Head Thread Issue
442706	01/17/11	This NCR Tracks SGT IIRP 146 Summary Of Work Related Issues
442711	01/17/11	This NCR Tracks SGT IIRP 147 Work Activities Ceased
442860	01/18/11	This NCR Tracks SGT DR 1024 Untimely Deviation Reporting
443077	01/19/11	This NCR Tracks SGT NCR 157 Missing Tendon Button Head
443290	01/20/11	This NCR Tracks SGT NCR 158 Tendon Wire Discrepancy
443343	01/20/11	This NCR Tracks SGT NCR 159 Broken Tendon Wire
443379	01/20/11	This NCR Tracks SGT NCR 160 Work Package Discrepancies
443424	01/20/11	This NCR Tracks SGT DR 26 Weld Rod Log Discrepancy
443487	01/21/11	Excess Leakage into the Tendon Access Gallery Sump
443529	01/21/11	Delay In Submitting SGT Qualification Cards
443563	01/21/11	Radio Repeater Power Loss Affected SGT Radio Communication
443566	01/21/11	SGT Workers Evacuated Tendon Galley Due To Exhaust Fumes
443587	01/21/11	FSP-2B Exhaust Disrupted Tendon Tensioning Work
443605	01/22/11	This NCR Tracks SGT NCR'S 162,163,164,165 Tendon 34V08 Issue
443606	01/22/11	This NCR Tracks SGT NCR 161 Missing Shim Spacer
443629	01/22/11	SGT Worker Put TLD Through NSOC X-Ray
443692	01/24/11	This NCR Tracks SGT DR 28 ILRT Configuration Control Deficiencies
443749	01/24/11	Bottles with Improper Fluids Discovered in Tendon Galley
443961	01/25/11	This NCR Tracks SGT NCR 166 Tendon Lift Off Pressure Deltas
443962	01/25/11	This NCR Tracks SGT IIRP 154 Hydraulic Leak
443966	01/25/11	This NCR Documents PE Observation 47866 Lanyard Not on Tool
444707	01/28/11	This NCR Tracks SGT IIRP156/DR 29 Lift Off Pressure Delta
444726	01/28/11	This NCR Tracks SGT NCR 167 Bent Tendon Shims
444728	01/28/11	This NCR Tracks SGT NCR 168 Water in Tendon Can
444971	01/31/11	This NCR Tracks SGT NCR 169 Water In Tendon Can
444972	01/31/11	This NCR Tracks SGT NCR 170 Broken Tendon Wire
445558	02/02/11	This NCR Tracks SGT NCR 171 Broken Wires 13H32 51H32 51H34
445761	02/02/11	FSAR Description Not Implemented During Containment Repair
445762	02/02/11	Tendon Detensioning Basis Doc Does Not Exist For Surveillance
445853	02/03/11	This NCR Tracks SGT NCR 172-IIRP 161 Shim Spacing Delta
445982	02/03/11	This NCR Tracks SGT NCR 173/ IIRP 162 Hydraulic Leak
446052	02/03/11	Laser Scan Data Is Outside of What's Expected
446084	02/04/11	This NCR Tracks SGT IIRP 163 Suspected Dropped Object
446225	02/05/11	This NCR Tracks SGT IIRP164 Dropped Object
446232	02/05/11	This NCR Tracks SGT IIRP 165 Broken Tendon Ram Gauge
446280	02/06/11	This NCR Tracks SGT IIRP 167 Vehicle Incident
446807	02/09/11	This NCR Tracks SGT NCR 174 Broken Wires 35H19 64H43 13H40
446808	02/09/11	This NCR Tracks SGT NCR 175 Stressing Sequence Discrepancy
447042	02/10/11	This NCR Tracks SGT NCR 176 Pass 5 53H21 Missing Wire

447376	02/12/11	This NCR Tracks Tendon 34V08 Issue
447415	02/13/11	Cannot Couple on Tendon 34V08
447416	02/13/11	Raised Metal Noted on Tendon 34V01 Anchor Head
447417	02/13/11	Protruding Buttonhead on Tendon 34V13
448236	02/17/11	This NCR Tracks SGT NCR 180-34V08 Dome End Foreign Material
448468	02/18/11	This NCR Tracks SGT IIRP 176 Oil Spill on Dome
448504	02/18/11	Tendon Tesile Testing Delta between SGR and Design Basis
448535	02/18/11	This NCR Tracks SGT NCR 183 "As Found" Shim Gap Delta
448537	02/18/11	This NCR Tracks SGT NCR 181 Documentation Deficiency
448579	02/19/11	This NCR Tracks SGT NCR 182 Bearing Plate Concrete Gap Delta
448580	02/19/11	This NCR Tracks SGT NCR 184 Tendon 56V13 Damaged Buttonhead
448581	02/19/11	This NCR Tracks SGT NCR 185 Bearing Plate Pitting 45V14
448621	02/20/11	Broken Wire On Tendon 45V01 SGT NCR 1186
448622	02/20/11	As Found Tendon Anchor Head Thread Damage 12V05,07,09,11,13
448639	02/20/11	SGT NCR 1188 Documents 23V20 Wire Delta
448640	02/20/11	NCR 1189 Pitting on 23V24 Bearing Plate
448659	02/21/11	This NCR Tracks SGT IIRP 178 Grease Hose Failure
449508	02/22/11	This NCR Tracks SGT IIRP 180 First Aid On Pinched Finger
449562	02/23/11	This NCR Tracks SGT NCR 190 Bearing Plate Concrete Gaps
450121	02/25/11	This NCR Tracks SGT NCR 191 Broken Tendon Wire 35H29
450133	02/25/11	This NCR Tracks SGT NCR 192 Bearing Plate Concrete Crack/Gap
450218	02/26/11	This NCR Tracks SGT IIRP 182 Harrington Hoist Not Working
450219	02/26/11	Tendon Gallery Bent Junction Box Cover SGT NCR 193
450591	03/01/11	This NCR Tracks SGT NCR 194 Tendon Elongation Delta Pass 8
451015	03/02/11	Worker Observed Standing on Platform 6 Hand Rail
451148	03/03/11	This NCR Tracks SGT NCR 197 As Found Flaking Coatings
451149	03/03/11	This NCR Tracks SGT NCR 198 Broken Wire 53H23 Shop End
451151	03/03/11	This NCR Tracks SGT NCR 201 Separated Existing Concrete
451166	03/03/11	This NCR Tracks SGT NCR 199 Broken Master Pressure Gauge
451167	03/03/11	This NCR Tracks SGT NCR 200 Gauge No "Post Use" Calibration
451306	03/03/11	This NCR Tracks SGT NCR 202 Protruding Wires Tendon 53H26
451412	03/03/11	Final Retensioning Using More Shims Than Anticipated
451425	03/04/11	This NCR Tracks SGT NCR 203 Pass 8 IWL Concrete Reject
451426	03/04/11	Strain Gage Alert Limit Exceeded By 0.3 Micro Strain
451471	03/04/11	This NCR Tracks SGT NCR 204 Bearing Plate Concrete Cracks
451549	03/05/11	This NCR Tracks SGT NCR 205 Broken Buttonhead 64H21
451632	03/06/11	This NCR Tracks SGT NCR 1206 Exposed Rebar Azimuth 200 Dome
451727	03/07/11	This NCR Tracks SGT NCR 207 Bearing Plate Concrete Gaps 53H3
451730	03/07/11	This NCR Tracks SGT NCR 208 Bearing Plate Concrete Gaps
451732	03/07/11	This NCR Tracks SGT IIRP 187 DROPPED OBJECT
451742	03/07/11	THIS NCR Tracks SGT NCR 209 Bearing Plate Concrete Gap 53H39
451900	03/07/11	This NCR Tracks SGT NCR 210 Bearing Plate Concrete Gaps
452030	03/08/11	IWL Inspection 51H41-35H41-53H41 Bearing Plate Concrete Crack
452032	03/08/11	This NCR Tracks SGT IIRP 190 Dropped Object

452178	03/08/11	This NCR Tracks SGT NCR 213 Anchorhead Delta 13H37 Field End
452188	03/08/11	IWL Inspection Bearing Plate Concrete Cracks 51H37 & 53H37
452577	03/10/11	This NCR Tracks SGT IIRP 193 Spalled Concrete Buttress 3-4
452578	03/10/11	IWL Bearing Plate Concrete Crack 62H30 Shop End
452718	03/10/11	This NCR Tracks SGT NCR 217 Pressure Gauge Delta
452820	03/11/11	This NCR Tracks SGT NCR 218 Broken Wire 64H34 Shop End
452822	03/11/11	IWL Bearing Plate Concrete Cracks 62H34
452826	03/11/11	SGT Craft Worker Hit Hand on Platform Scaffold Knuckle
452954	03/12/11	IWL Tendon Bearing Plate Concrete Crack 62H37
452960	03/12/11	This NCR Tracks SGT IIRP 198 Small Hydraulic Spill
452970	03/12/11	IWL Tendon Bearing Plate Concrete Cracks 62H33
453016	03/12/11	Conservative Call during Acoustic Monitoring
453037	03/13/11	IWL Tendon Bearing Plate Concrete Crack 62H40 & 62H36
453038	03/13/11	Horizontal Tendon 64H43 Field End Anchor Head Delta
453054	03/13/11	Conservative Stop Work for AE Monitoring
453079	03/13/11	Embedded Wood in Feature Strip Bay 5-6 El 240'
453139	03/14/11	IWL Tendon Bearing Plate Concrete Cracks 62H39
453140	03/14/11	This NCR Tracks SGT NCR 225 Anchor Head Delta 64H44
453318	03/14/11	Retensioniing Work Stopped for AE in Bay 5-6
453368	03/15/11	IWL Tendon Bearing Plate Concrete Crack 62H41
453371	03/15/11	This NCR Tracks SGT NCR 227 Mistras Monitoring Limits Exceed
454461	03/21/11	This NCR Tracks SGT DR 33 ANII Not Offered Review of WPCN
454525	03/21/11	This NCR Tracks SGT DR 32 Concrete Monitoring Delta
455744	03/28/11	Spalled Concrete Discovered In Bay 5-6 Panel J
457461	04/04/11	This NCR Tracks SGT NCR 228 Spalled Concrete Bay 2-3 Panel J
458265	04/07/11	This NCR Tracks SGT NCR 229 Spalled Concrete Bay 2-3
458925	04/11/11	This NCR Tracks SGT NCR 230 Mistras Acoustic Equipment Issue
459678	04/14/11	Containment Tendons Greasing Requirements Were Not Met
460175	04/18/11	This NCR Tracks SGT NCR 231 Damaged Floor Coatings
460177	04/18/11	This NCR Tracks SGT NCR 232 Tendon Greasing Delta
460479	04/19/11	Horizontal Indication In Containment Structure Wall
460491	04/19/11	Tendon Greasing Requirements Have Not or Will Not Be Met

Drawings
75221-103, SIT Instrument Locations – Elevation 103’-0”
75521-135, SIT Instrument Locations – Elevation 135’-0”
75521-170, SIT Instrument Locations – Elevation 170’-0”
75521-205, SIT Instrument Locations – Elevation 205’-0”
75521-240, SIT Instrument Locations – Elevation 240’-0”
75521-250, SIT Instrument Locations – Elevation 250’-0”
75521-DOME, SIT Instrument Locations – From Reactor Building Dome
75521-EH, SIT Instrument Locations – Elevation 135’-0”
S-425-004, Revision 1, IWE/IWL Inspection Vertical Tendon Layout
S-425-005, Revision 1, IWE/IWL Inspection Hoop Tendon “13” Layout
S-425-006, Revision 1, IWE/IWL Inspection Hoop Tendon “42” Layout
S-425-007, Revision 2, IWE/IWL Inspection Hoop Tendon “53” Layout
S-425-008, Revision 1, IWE/IWL Inspection Hoop Tendon “64” Layout
S-425-009, Revision 1, IWE/IWL Inspection Hoop Tendon “51” Layout
S-425-010, Revision 1, IWE/IWL Inspection Hoop Tendon “62” Layout

Z08, Revision 9, Tendon Tensioning

Other Documents
Engineering Change 75221, Revision 3
Extensometer Installation Specifications and Guidance

LIST OF ACRONYMS USED

ACI	American Concrete Institute
ADAMS	Agencywide Documents Access and Management System
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing Materials
CFR	Code of Federal Regulations
CR3	Crystal River Unit 3
EC	Engineering Change
FAC	Flow-Accelerated Corrosion
ft	foot (feet)
HTS	Horizontal Transfer System
ILRT	Integrated Leak Rate Test
IP	Inspection Procedure
IR	Impulse Response
MT	Magnetic Particle Testing
NCR	Nuclear Condition Report
NCV	Non-Cited Violation
NDE	Non-Destructive Examination
NRC	Nuclear Regulatory Commission
NUREG	Publications Prepared by the NRC staff or contractors
OLS	Outside Lift System
OSGs	Original Steam Generators
PARS	Publically Available Records
PT	Liquid Penetrant Testing
QA	Quality Assurance
RCS	Reactor Coolant System
RFO	Refueling Outage
ROTSGs	Replacement Once-Through Steam Generators
RSG	Replacement Steam Generator
RT	Radiographic Examination
SCV	Steel Containment Vessel
SCLP	Steel Containment Liner Plate
SG	Steam Generator
SGR	Steam Generator Replacement
SGRP	Steam Generator Replacement Project
SIT	Special Inspection Team and Structural Integrity Test
SPMT	Self-Propelled Modular Transporter
TLD	Temporary Lifting Device
UFSAR	Updated Final Safety Analysis Report
UT	Ultrasonic Examination
WO	Work Order
WP	Work Package

Enclosure 2 Request and Response Database

NRC inspectors gather information through a wide variety of methods, including personnel interviews, record reviews, and direct observations. During this process, inspectors may ask questions to gain additional or clarifying information. The licensee maintained a database to track NRC inspection questions and their responses to those questions. The enclosed database reflects some of the questions that the inspectors asked and some of the information reviewed during the course of inspection. Some of the licensee responses refer to calculations, procedures, etc., which resided outside of the database and may have been subject to separate review. The database has been enclosed in this report because the licensee's responses contained unique technical information important to the NRC's understanding of the containment delamination issue and repairs, and because this information may not otherwise be retrievable. At the end of this inspection period, this database remained an open document subject to periodic updates by the licensee as new questions were captured or when new information became available.

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:34 AM

Request Number: 1

Individual Contacted: Sid Powell

Date Contacted: 10/15/2009

Requestor/Inspector: Louis Lake

Category: Question

Request: There is an area on the containment dome on the south, approximately half way between the walking platform and the peak of the dome that is depressed. There appears to be a grout covering that is seriously deteriorated. Is this evidence of repeat delamination damage?

References:

Response Assigned to: Craig Miller

Date Due to Inspector: 10/16/2009

Response:

I was up on the dome earlier this evening to examine the entire dome structure since I also have a Pri 3 investigation upcoming regarding the condition of the concrete on the dome (reference AR 357670). Although it had been a number of years since my last visit up there, the overall condition of the dome is pretty much exactly the same as it has been in my past trips as part of tendon surveillance. I believe that when they made the re-pours of the dome due to the original delamination, the final surface did not end up being a smooth arcing curvature and had several localized uneven areas. The one in question is exactly that.

Furthermore, as part of our ongoing Condition Monitoring of Structures effort (EGR-NGGC-0351), I will be returning to the dome this evening (10/16/2009) with Dayna Mendez to obtain digital photographs of the area to insert into our data base on this subject so that we have a reference point for future inspections.

Additional data: An action request was initiated on September 29, 2009 (AR 357670) which stated, "Concrete spalling has been noted on the dome of the reactor building roof. An evaluation needs to be performed for long range plan purposes." The evaluation was documented in assignment 18 (AR 357670-18) and was approved on October 21, 2009. The evaluation is included in this folder.

A copy of the Construction Microfiche log is included here:

L:\Shared\CR3 Containment\ROOT CAUSE ANALYSIS Files\(2) Concrete Construction\Construction MicroFiche Index.pdf

Misc Notes:

Response By: Joe Lese

Reviewed By:

Date Response Provided: 10/16/2009

Status: Closed

Date Closed: 1/28/2010

20-Apr-11 7:56:35 AM

Request Number:	2
Individual Contacted:	Sid Powell
Date Contacted:	10/16/2009
Requestor/Inspector:	Anthony Masters
Category:	Information Request
Request:	The Inspector has requested a procedure that was used for tensioning the tendons originally.
References:	

Response Assigned to:	Sid Powell
Date Due to Inspector:	10/16/2009

Response:

Prescon Field Installation Manual.tif was placed in folder L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\POWELL Q-A\Request 2, Original Tendon Tensioning Procedure

Misc Notes:

Response By:	Sid Powell
Reviewed By:	
Date Response Provided:	
Status:	Closed
Date Closed:	2/25/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 3

Individual Contacted: Paul Fagan

Date Contacted: 10/16/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: George Thomas of the NRC requested a copy of our EC 74801 on the core bores. I printed a copy and delivered to Mr. Thomas at 0900. This is a preliminary copy since the EC has not been issued yet.

References:

Response Assigned to: Glenn Pugh

Date Due to Inspector: 10/16/2009

Response:

An approved (issued) copy of Revision 1 of the EC 74801 was printed and hand carried to George Thomas on 10/28/09.

Misc Notes: See question #19 for continued submittal of NDE and Core Bore Plan changes.

Response By: Glenn Pugh

Reviewed By: Charles Williams

Date Response Provided: 10/28/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 4

Individual Contacted: Sid Powell **Date Contacted:** 10/16/2009

Requestor/Inspector: George Thomas **Category:** Information Request

Request: Please provide the concrete mix design and associated material test data for concrete use in original construction of the containment wall. Also provide original test data of production concrete used in the original construction of the containment wall.

References: Calculation S00-0047

Response Assigned to: Sid Powell **Date Due to Inspector:** 10/17/2009

Response:

The RB exterior shell consisted of around 105 separate concrete pours. Attachment B of calculation S00-0047 shows a listing of these pours by elevation and buttress zone. It also lists the mix design for each pour. For example, the SGR containment opening is between buttress 3 and 4 and between Elevations 180’ and 220’. Per the pour list in the calculation the corresponding pour numbers are 685RB, 695RB, 700RB, 712RB, and 722RB. The construction microfiche listing then gives a corresponding microfiche card number for each of these pours. For example the records for pour number 685RB are on card 1P08022. A typical microfiche card will contain several pages of information including the mix design, batch tickets (truck slips), the date of the pour, curing data, and other relevant data. CR3 Document Services are attempting to scan these cards for use by the NRC and Root Cause team. At this time, there are some examples of the pour cards at L:\Shared\Containment Root Cause Files\Requested by NRC. A copy of calculation S00-0047 is also included at this location. Document Services is attempting to scan the pours between buttresses 3 and 4 (all elevations) first. If a different location is required, please let Glenn Pugh

C. G. Pugh 10/17/09

Misc Notes:

Response By: Glenn Pugh

Reviewed By: **Date Response Provided:** 10/16/2009

Status: Closed **Date Closed:** 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 5

Individual Contacted: Sid Powell

Date Contacted: 10/16/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: With regard to the SGR Construction Opening, please provide stress plots of the SGR Opening and surrounding areas for the Dead load + Prestress load combination for the following cases: (i) prior to tendon detensioning and removal (ii) after tendon removal; (iii) with SGR opening and (iv) After restoration of opening and tendon retensioning.

References:

Response Assigned to: Sid Powell

Date Due to Inspector: 10/17/2009

Response:

George asked if we could provide stress plots for the analysis at the SGR opening for the Dead Load + Pressure Load combination at the 4 stages of the SGR project. Unfortunately we did not run computer stress analyses for the various load combinations. Each load element (dead load, pressure, liner plate thermal, thermal gradient, etc.) were individually evaluated. Additionally each were run at unit values, as to support the various amplification factors applied to the design basis evaluations. The results of these analyses were then extracted from the structural analysis package and processed, as necessary, to address the load combinations for various building conditions throughout the outage. Unfortunately, the program used does not have the ability to develop stress plots.

Misc Notes: Response inadequate. By this question, the NRC is seeking information to understand the structural behavior and response of the Containment Wall under real loads (i.e., Dead + applicable Prestress Load) in and around the SGR construction opening area for the configurations prior to, during and following creation of the SGR construction opening. Provide the pertinent information in an easily reviewable form. This information may be provided with pending response to Question 28.

Response By: Dan Jopling

Reviewed By:

Date Response Provided: 10/20/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	6		
Individual Contacted:	Sid Powell	**Date Contacted:**	10/16/2009
Requestor/Inspector:	Dan Naus	**Category:**	Question

Request: Were the vertical and hoop tendons in the SGR opening area subject to lift-off measurements before detensioning and removal. If so provide lift off measurements. Were the removed tendons inspected/examined and if so what were the findings.

References:

Response Assigned to:	Sid Powell	**Date Due to Inspector:**	10/16/2009

Response:

No lift off measurements were made for the tendons that were removed from the opening.

IWL examinations were performed on the concrete and bearing plates for the removed tendons. tendon end examinations were performed on the two longest tendons that were non-destructively removed. One wire each was removed and examined for the two longest tendons.

Misc Notes: Does CR3 plan on performing tension testing (i.e., ultimate strength, yield strength and elongation) on a wire sample from one or more of the removed hoop tendons that exhibited higher than anticipated loss of prestressing force (i.e., hoop tendons that did not meet the 95% predicted value criteria in IWL)? This information may be provided with pending response to Question 22.

Response By:	Sid Powell		
Reviewed By:		**Date Response Provided:**	10/16/2009
Status:	Closed	**Date Closed:**	1/28/2010

Request Number: 7

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Dan Naus

Category: Information Request

Request: Provide de-tensioning sequence in R16 for the construction opening. Provide procedure? Did anyone hear anything?

Follow up request: Documents related to the dome delamination seem to indicate that a loud noise or boom was heard on December 4, 1974, however, no noticeable damage was observed during a subsequent visual inspection. Did anyone hear a loud noise or boom during the detensioning procedure related to the SGR construction opening?

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

R16 Tendon Detensioning sequence.pdf: {E-mail from the SGR Tendon Field Engineer on the detensioning sequence.}

Containment Opening - Tendon Removal Timeline.xlsx: {Spreadsheet containing some interview questions and responses as well as some plant shutdown/mode times and tendon detensioning sequence information.}

Z3R5 PSC Field and Quality Control Manual1.pdf: {PSC Procedures [ALL], F&Q 8.0, 8.1, and 10.0 specifically address Tendon Detensioning/Removal, Plasma Tendon Detension, and Tendon Removal}

Follow up Response: Interviews were performed with craft and supervisory personnel associated with detensioning and hydroblasting. None indicated any abnormal noises occurring during these evolutions. Some were asked specifically if any loud noises were heard and no one identified any abnormal loud noises. Additionally, seismic monitoring data was obtained and reviewed for indication of movement. WO 1654188-01 shows no evidence of movement. Note: One direction was invalidated due to disturbance that occurred during data retrieval. The other two directions showed no movement. See Seismic Data - PT-379.pdf file at L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 7, Q1 Response Info - Portmann

Misc Notes:

Response By: Rick Portmann / Charles Williams

Reviewed By:

Date Response Provided: 11/2/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 8

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Dan Naus

Category: Information Request

Request: Any information on significant repairs (concrete related) between buttress 3 and 4 from original construction to today.

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

Performed a search of the document control system, both the SEEK system and historical QA records. Looked for any Work Orders, NCRs, Correspondence, or other documents using the keywords “concrete repair” and “concrete crack.” There were several “hits” on these key words. The majority of these “hits” were screened away by reviewing the title of the document. Any “hits” where the title was not clear were reviewed individually. The results were several AR’s and Work Orders to repair damaged or cracked concrete on the RB containment. However, none of the items reviewed were in the area of concern. Document search summaries are here: L:\Shared\CR3 Containment\NRC SIT Team Questions & Info\Request 8, Q2 Response Info- Pugh

In addition, conversations were held with several people in maintenance and engineering, including one person that was employed in the early 1970’s. No one could remember making any repairs on the RB shell concrete in the area of interest. No modifications could be identified. Conclude that the concrete between buttress 3 and 4 is original construction.

Misc Notes:

Response By: Glenn Pugh

Reviewed By: Charles Williams

Date Response Provided: 10/28/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	9		
Individual Contacted:	Garry Miller	**Date Contacted:**	10/22/2009
Requestor/Inspector:	Dan Naus	**Category:**	Question

Request: Was there any analysis of why re-tensioning was required in past tendon surveillance activities (done at that time of surveillance testing)?

Follow up request: Since lower than expected lift-off loads have been obtained in the recent 3 tendon surveillances for a significant number of horizontal tendons, describe your plan, if any, to determine, evaluate and eliminate the cause(s) of the condition not meeting the IWL acceptance by examination criteria.

Follow up request: Is the cause of the larger than anticipated losses of prestressing force in several hoop tendons being addressed as part of the root cause assessment?

References:

Response Assigned to:	Charles Williams	**Date Due to Inspector:**	10/26/2009

Response:

There was no analysis performed during past surveillance testing years in which tendons were re-tensioned.

Additional information in response to the above question: See License Request No. 24 – NRC SIT Question# 18 folder, under sub-folder: “IWL - Tendon Surveillance History” for information, discussions and actions taken related to tendon lift-off testing and re-tensioning.

Misc Notes:

Response By:	Rick Portmann		
Reviewed By:		**Date Response Provided:**	11/2/2009
Status:	Closed	**Date Closed:**	7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 10

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Dan Naus

Category: Question

Request: When CTL is using IR and IE, can they determine relative concrete quality of locations tested as part of CTL NDE procedures?

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 10/26/2009

Response:

In general, the Impulse Response (IR) test results is influenced by concrete quality and existence of defects at the test point. The aspects in concrete influencing IR results include presence of delamination, cracking, significant void or honeycomb and change in concrete properties. The most significant factor is the presence of delamination which effectively reduces the thickness of wall or slab responding to the impact. Considerable difference in quality of concrete is typically reflected in the test results. For example, a core removed from panel RBCN-0014-N (Core #13) where a higher mobility value was obtained by NDT, had less coarse aggregate in the concrete, which changed density and modulus in that localized area, no delamination was noted in these areas with subsequent boroscope examinations.

Misc Notes:

Response By: Paul Fagan

Reviewed By:

Date Response Provided: 11/12/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 11

Individual Contacted: Garry Miller **Date Contacted:** 10/22/2009

Requestor/Inspector: Louis Lake **Category:** Question

Request: Does the PGN Testing Procedure identify how CTL calibrates their equipment, qualification of personnel, and equipment set-up (i.e., frequencies)? Provide Testing Procedure to NRC.

References:

Response Assigned to: Paul Fagan **Date Due to Inspector:** 10/26/2009

Response:

This question pertains to PGN procedure PT-407T, Reactor Building Concrete Examination and Testing, Revision 2.
The question is split into three areas with specific procedure steps stated to address each area.
Area 1 – Calibration
Step 3.2 Responsibilities
Step 3.2.1
The Condition Assessment Consultant is responsible for:
Provide equipment list and associated calibration documentation
Step 3.3 Limits & Precautions
Step 3.3.2
The equipment utilized to perform the NDT was calibrated in the field during trial use by CTLGroup. This method of validating the test process and equipment for a specific application is standard practice for concrete condition assessments utilizing NDT.
Step 5.3 Reports
Step 5.3.1
An equipment list with calibration documentation will be provided for the NDT used. The NDT process calibration/validation document will be included in the report.
Enclosure 7
For a critical structure of this scale, more correlation data is desired in order to finalize a more comprehensive calibration.
Enclosure 8
Individual equipment packages have been established to track specific calibrated equipment in order to link individual NDT locations with a calibrated equipment package. The Exterior Containment Inspection Log requires an Equipment Package Number to be recorded for each NDT location. The Equipment Package Number is traceable to a permanent plant record documenting the calibration records for the equipment.

Area 2 – Qualification
Step 3.2 Responsibilities
Step 3.2.1
The Condition Assessment Consultant, CTLGroup, shall be responsible for assuring that all individuals under his supervision are properly trained in the use of this procedure and associated equipment.
Step 3.2.1
The Condition Assessment Consultant is responsible for:
Provide personnel qualification records for lead Engineer
Step 3.5.2 Initial Conditions
ENSURE that all personnel are familiar with the operating manuals of the equipment to be used during the inspection.

Step 5.3 Reports
Step 5.3.1
The report will include personnel qualification records of lead engineers who performed the NDT.
Area 3 – Equipment set-up
Step 3.2 Responsibilities
Step 3.2.1
The Condition Assessment Consultant is responsible for:
Provide calibration/validation documentation to substantiate the NDT methods to be used and to support the dedication of the software (SMASH) being used to evaluate the NDT data.
Step 3.3 Limits & Precautions
Step 3.3.2
The equipment utilized to perform the NDT was calibrated in the field during trial use by CTLGroup. This method of validating the test process and equipment for a specific application is standard practice for concrete condition assessments utilizing NDT.
Enclosure 5, page 1
TURN ON the computer to start setup process.
Enclosure 6, page 1
TURN ON the computer to start setup process.

Misc Notes:

Response By: Paul Fagan

Reviewed By:

Date Response Provided: 11/12/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 12

Individual Contacted: Garry Miller **Date Contacted:** 10/22/2009

Requestor/Inspector: Dan Naus **Category:** Question

Request: Once the construction opening is refilled with concrete, how and for how long will the concrete be allowed to cure, and what is decision process for start of post-tensioning the structure?

Follow up request: In light of the apparent much more extensive repair area affected by delamination, how will the concrete curing and decision process for start of post-tensioning be affected?

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:** 10/26/2009

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\DYKSTERHOUSE Q-A

•Concrete will be cured for 7 days from the time of placement (Ref. 1).
• After forms are stripped a curing compound is applied (Ref. 1)
•Forms may be removed after 3 days, or sooner if the concrete has achieved a compressive strength of 3000 psi as demonstrated through strength testing (Ref. 1)
•Tendon retensioning starts with the verticals at buttress #3 and #4 (23V1 thru 23V3, 45V22 thru 45V24) after the concrete reaches a compressive strength of 5000 psi, followed by the remaining verticals outside the opening (34V18 thru 34V24, 34V1 thru 34V7) in parallel with the hoop tendons above and below the opening (42H22 thru 42H26, 53H23 thru 53H26, 42H35 thru 42H39 and 53H36 thru 53H39). After the concrete reaches 6000 psi the tendons within the opening are retensioned (34V8 thru 34V17 and 42H27 thru 42H34 and 53H27 thru 53H35). Tendon retensioning sequence is shown in detail on drawing 421-352 (Ref. 2).
The following is extracted from Ref. 3, page 86 and provides concrete mix strength information that may support removing the formwork earlier:
The use of autogenous curing containers is not planned during the containment opening concrete placement. Although autogenous containers would better represent the curing environment before formwork removal, their use involves additional resources and storage space. Therefore, standard curing methods will be used during actual concrete placement at the opening. To better understand what the difference in compressive strengths would be between the two methods, S&ME was tasked with testing a batch of concrete (concrete proportions based on results of Phase II testing i.e. Option 1A) and determining the difference in compressive strength between the two curing methods at 1, 2, 3, 5 and 28 days.
Attachment Z55R3 contains the S&ME test methodology and test results. Test results for compressive strength are reproduced below:
Age, daysAutogenous ContainersAlternative Proposed CuringAutogenous/Alt
15,620 psi4,760 psi18% increase
26,4505,9309% increase
36,5906,3204% increase
56,8606,8300%
288,0508,4805% decrease
The results clearly indicate that the autogenous cured cylinders have higher early age strength which is as expected. The heat of hydration is (to some extent, over and above the standard cylinders) trapped inside the containers resulting in a harder concrete. These results would indicate that formwork could be removed as early as one day after completion of concrete placement.
Per Ref. 1:

Specified Concrete Strength: 6000 psi at 5 days, 7000 psi at 28 days
Slump: 6” to 9”
Air Entrainment: 0% to 3.5% maximum
Concrete Unit Wt: 145 pcf minimum

References:
1.Specification CR3-C-0003, Rev. 0, Specification for Concrete Work for Restoration of the SGR Opening in the Containment Shell.
2.Drawing 421-352, Rev. 0, RB Temporary Access Opening for SGR – Restoration – Sheet 1 of 1
3.EC 63016, Rev. 26, Containment Opening

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 13

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: Before additional tendons are de-tensioned, will there be as-found lift off measurements taken for these tendons.

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

Planning and scheduling are currently in progress to obtain lift-off measurements of some of the tendons which are going to be detensioned. The root cause team has requested lift-off data on vertical tendons 34V3 thru 34V7 & 34V18 thru 34V22 and horizontal tendons 42H22 thru 42H26 & 42H35 thru 42H39.

See lift-off data provided in Request 6 response.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/4/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	14		
Individual Contacted:	Garry Miller	**Date Contacted:**	10/22/2009
Requestor/Inspector:	Dan Naus	**Category:**	Question

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

References:

Response Assigned to:	Charles Williams	**Date Due to Inspector:**	10/26/2009

Response:

Section 5.3.2 of the Dome Repair report included with Letter 3F1276-10 outlines where the strain gauges were attached.

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

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

Documents for this response are located here: L:\Shared\CR3 Containment\NRC SIT Team Questions & Info\Request 14, Q8 Response Info- Pugh

Misc Notes:

Response By:	Glen Pugh		
Reviewed By:	Charles Williams	**Date Response Provided:**	10/28/2009
Status:	Closed	**Date Closed:**	1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	15		
Individual Contacted:	Garry Miller	**Date Contacted:**	10/22/2009
Requestor/Inspector:	Louis Lake	**Category:**	Question

Request: When the 1976 roof delamination issue occurred, was there any evaluation of the rest of containment, including a “notch sensitivity” review? Refer to the FPC Final Report Page # 110. a) was the concrete different in the containment versus the dome?

References:

Response Assigned to:	Charles Williams	**Date Due to Inspector:**	10/26/2009

Response:

To answer the first part of the request a review of the Delamination Report with subsequent supplements and letters was completed. Could not find an indication that a “notch sensitivity” review was performed on the remaining portions of containment.

The answer to the second part of the request is outlined here:

•Calculation S00-0047 contains a summary of the concrete pours for the containment structure. The containment walls (between buttresses) were poured using one of two Class 5000 psi mix designs, DM-5 or 727550-2.

•Calculation S00-0047 also shows the concrete pours prior to the dome delamination and the concrete mix design to repair the dome delamination. The concrete mix in the original dome was the same 727550-2 mix as the walls. The repaired portions of the dome used concrete mix 658550-2. This newer mix was 6000 psi class concrete.

Misc Notes:

Response By:	Glenn Pugh		
Reviewed By:		**Date Response Provided:**	6/8/2010
Status:	Closed	**Date Closed:**	6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 16

Individual Contacted: Garry Miller **Date Contacted:** 10/22/2009

Requestor/Inspector: Louis Lake **Category:** Information Request

Request: Discuss the planned NDE method, its reliability, industry experience, and other pertinent information.
B) Discuss supplementary verification plans to ensure results are reliable.

References:

Response Assigned to: Paul Fagan **Date Due to Inspector:** 10/26/2009

Response:

A) Impulse Response (IR) test was chosen as the primary NDT technique to evaluate the extent of delamination. The IR method uses a low strain impact from a hammer equipped with a load cell to send a stress wave through the element under test. The response to the input stress is measured using a velocity transducer (geophone). Both the hammer and the geophone are linked to a portable field computer for data acquisition and storage. Time records for both the hammer force and the geophone velocity response are transformed into the frequency domain using the Fast Fourier Transform (FFT) algorithm.
Average Mobility is the key parameter that the dynamic IR test produces. It is defined as the structural surface velocity responding to the impact divided by the force input [(m/s)/N]. The mean mobility value over the 0.1-1 kHz range is directly related to the modulus, density and the effective thickness of the element. In general, presence of significant voiding or an internally delaminated or un-bonded layer will result in an increased average mobility value. On the other hand, a sound concrete element without distress will produce a relatively low average mobility value. The test results can be analyzed and presented in the form of contour plots. The suspect areas can be identified through a scaled color scheme.
Comparing to another well-known NDT method Impact-Echo (IE) test, the IR test uses a compressive stress impact approximately 100 times that of the IE test. This greater stress input means that the plate responds to the IR hammer impact in a bending mode over a very much lower frequency range (0-1 kHz for plate structures), as opposed to the reflective mode of the IE test which normally requires a frequency range of approximately 5 to 30 kHz, The influence of reinforcement and tendons in the structure has generally less impact than it would for IE test, while delamination at relatively shallow depth, if any, will dominate the signal response in IR testing. It makes it ideal to evaluate the presence of delamination without having to layout locations of tendon and reinforcing bars prior to the testing in a time critical project. However, the IR test cannot detect with high certainty the absolute depth of delamination; rather it's on a comparative basis. The width or size of crack cannot be determined in the IR testing.
The IR test method has been used to evaluate concrete structure condition in the past 20 years. The test method is in the process of being standardized by ASTM. CTLGroup has extensive experiences in utilizing this method to characterize defects in concrete. IR test has been used in evaluating concrete structures in both nuclear and fossil power plants. CTL Group experience for nuclear related structures has been compiled (see attached).
B) According to the Progress Energy procedure PT-407T, Rev. 2, concrete core samples are removed in areas with high mobility values (greater than 1.0) to confirm the presence of delamination. Core samples are also removed in areas where mobility value is in the "Gray" (between 0.4 and 1.0) range to verify the condition, unless the slightly elevated values can be dispositioned through evaluation. Many cores have been removed based on the IR test results along the boundary of delamination in the section where steam generator opening is located. At this time, the approximate 20 cores so far removed indicated the IR results have been accurate in characterizing the extent of delamination in the steam generator opening area. Also according to the test procedure, a population of core samples is also removed from areas where low mobility values (less than 0.4) are obtained to confirm the sound concrete condition. Based on the core samples removed, the IR results have been accurate to detect a delamination in the concrete.

Misc Notes:

Response By: Paul Fagan

Reviewed By:

Date Response Provided: 11/12/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

Request Number: 17

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: For petrographic analysis, who are the labs and what are their credentials?

Follow up Request: Provide information on the qualification of the petrographers from CTL and Photometrics who are performing/supervising petrographic examination work for CR3.

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 10/26/2009

Response:

Two labs have performed petrographic analyses in accordance with ASTM C 856: MACTEC Engineering & Consulting and CTL Group. MACTEC performed petrographic analysis under their Appendix B program, while CTL performed an informational "comparison" analysis as an additional, independent data point. The resume and qualification package of the Mactec individual who performed the analysis for CR3 is attached, as well as the CTL analyst's resume and petrography literature from the CTL website. A third laboratory, PhotoMetrics, is also performing material analysis, although not per the ASTM standard. The material examinations being performed by Dr.Mostafa at the PhotoMetrics laboratory involve methods intended to examine similar conditions and attributes evaluated under petrographic examinations, but using tools and techniques more frequently used in material science, e.g., scanning electron microscope (SEM) and micro-hardness examinations that are more thorough. Information from the PhotoMetrics website is attached.

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 11/18/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 18

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Dan Naus

Category: Question

Request: How are core samples being processed and sent to the labs for petrography?
A) How will you determine that the results are consistent between the labs?

Follow up Request: Please expand your response on the question of determining consistency of results between the labs. This may be provided with response to new question __ below.

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 10/26/2009

Response:

Each of the cores used for petrographic analysis was obtained with a 4” diamond core bore bit, sealed in aluminum foil and plastic, wrapped in bubble wrap, and packaged in wooden crates. The packages were shipped via FedEx for overnight delivery. Chain of Custody forms are used to track each core. Cores #5 and #7 were sent to MACTEC for analysis. MACTEC cut core #5 longitudinally and sent half to CTL. Core #6 was sent to PhotoMetrics using the same process.
The labs are each performing independent analyses. The primary goal of the analyses was to estimate the relative age of the cracked surface. Each lab was given this objective when the work was authorized. Final reports will be issued with results.

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 11/18/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 19

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: What is the sampling plan for NDE and core samples,

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 10/26/2009

Response:

The nondestructive testing (NDT) and core bores are being executed based on the requirements specified by the Root Cause Team in support of the root cause analysis, design basis evaluation, and repair requirements. NDT is performed on the exposed surfaces of the containment in each of the six bays, where a bay is defined as the area between each of the six buttresses. NDT is also planned to be performed on the dome surface and is in progress on the containment walls accessible from within adjoining buildings such as the Auxiliary Building, Intermediate Building, and the Fuel Transfer Building.

Exposed Surfaces

Exposed surfaces accessed via work platforms, scaffolding, ladders, and roofs of adjoining buildings are included in the condition assessment of structure. A small percentage of the overall surface area of exposed surfaces has physical constraints that make access impractical.

Adjoining Building Surfaces

Surfaces within adjoining buildings are accessed via permanent platforms, scaffolding, and ladders included in the condition assessment of the structure. A large percentage of the accessible surfaces are included in the plan; however, physical constraints exist in each of the three adjoining buildings that limit access. Examples are 1) areas with wall attachments that limit access to the concrete surface, 2) locked high radiation areas, and 3) contaminated areas.

Core Bores

The location and number of core bores is defined by the on-going NDT results and input from the Root Cause Team. Core bores are taken to provide samples for concrete testing. Cores in both solid and delaminated areas characterized by NDT are used to confirm the test results. Core bores have been drilled around the perimeter of the delamination in the bay between buttresses 3 and 4 to confirm the boundary of the delamination characterized by NDT.

Misc Notes:

Response By: Paul Fagan

Reviewed By:

Date Response Provided: 11/12/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 20

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: What are your examination plans for below grade?

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 10/26/2009

Response:

The containment exterior concrete surfaces not exposed to the elements are accessed from within the Auxiliary, Intermediate, and Fuel Transfer Buildings. The containment wall rests on the foundation mat. The top surface of the foundation mat is at EL. 93'-0" (ref. drawing 421-004). No portion of the containment wall is inaccessible due to concrete being in contact with backfill (below grade). Surfaces within adjoining buildings are accessed via permanent platforms, scaffolding, and ladders are included in the condition assessment of the structure. A large percentage of the accessible surfaces are included in this assessment; however, physical constraints exist in each of the three adjoining buildings that limit access. Examples are 1) areas with wall attachments that limit access to the concrete surface, 2) locked high radiation areas, and 3) contaminated areas.

Misc Notes:

Response By: Paul Fagan

Reviewed By:

Date Response Provided: 11/12/2009

Status: Closed

Date Closed: 1/12/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 21

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Provide interview observations from personnel involved with hydro-demolition and detensioning/cutting of tendons (when their comments note something of interest).

Provide information from additional interviews of personnel when they become available. Also, include interviews conducted by PII.

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #21, Q15 Response Info - Portmann

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 2/25/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 22

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: Have removed tendons been inspected and were there any significant findings?

Does CR3 plan on performing tension testing (i.e., ultimate strength, yield strength and elongation) on a wire sample from one or more of the removed hoop tendons that exhibited higher than anticipated loss of prestressing force (i.e., hoop tendons that did not meet the 95% predicted value criteria in IWL)?

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

There was no requested/required inspections performed of the removed tendons. Various questions were asked of the SGR Tendon Field Engineer and PSC Lead Individual, responses documented in the enclosed.
{Containment Opening - Tendon Removal Timeline.xlsx}
{10 28 interview Cliff Peters Gary Goetsch.pdf}
{Interview with Gary Goetsch.pdf}

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/2/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 23

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request:
When were observations of surface feature changes and water leakage noted below the construction opening?

At what location of the SGR opening area did hydro-demolition begin and what was the sequence of progression for the creation of the opening?

Provide a copy of NCR 358724 that identified voids in the RB concrete in the area of hydro-demolition.

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

The below is the timeline of events as noted in the Outage Autolog system (relevant Autolog pages attached) :
10/1/2009 4:28:59 AM Begin hydro-demolition
10/1/2009 1:15:08 PM Hydro-demolition to first layer of rebar is complete, begin cutting rebar
10/2/2009 3:55:53 AM Restart hydro-demolition
10/2/2009 5:15:30 AM Stream of water identified exiting RB wall from below/to the right of the transfer opening. Hydro-demolition suspended.
10/2/2009 6:41:11 AM Voiding identified in RB wall
10/7/2009 12:52:15 PM 2 ft x 4 ft loose concrete below the containment opening.

Copy of NCR 358724 also provided in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 23, Q17 Response Info - Miller

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 11/18/2009

Status: Closed

Date Closed: 2/3/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 24

Individual Contacted: Garry Miller **Date Contacted:** 10/22/2009

Requestor/Inspector: George Thomas **Category:** Question

Request: What were results of the last three IWE/IWL surveillance reports (provide actual complete reports)?
Provide inspection procedures and including qualification of personnel information?

References:

Response Assigned to: Charles Williams **Date Due to Inspector:** 10/26/2009

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 24, Q18 Response Info-Portmann

Misc Notes: Added the 2009 IWE Visual Examination reports to the NRC Folder for Question #24, per NRC verbal request. (Rick Portmann)

Response By: Charles Williams

Reviewed By: **Date Response Provided:**

Status: Closed **Date Closed:** 2/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 25

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Provide results of current visual inspections.

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 10/26/2009

Response:

In response to your E-Mail Clarification on 11/30/09 for information regarding "all IWL examinations, being performed during this (R16) outage, "to let you know there were no scheduled As-Found IWL examinations for this outage as they are performed every 5 years and were performed last in outage R15 (2007) [that information has been provided to you under NRC Folder "WILLIAMS Q-A" file "Request 24, Q18 Response Info-Portmann"]. The only IWL examinations scheduled are the As-Left Pre-Service IWL exams to be performed prior to, during, and following the ILRT on the repair/replacement area which is yet to be completed.
However as a result of the containment crack we did an augmented IWL scope between buttresses 3-4 to compare to the R15 information as part of the root cause investigation. I have included these reports, reference file RO-16 IWL Exam Reports.pdf enclosed in the NRC folder "FAGAN Q-A" file "Request 25, Q19 Response Info".
The SGR-QC also performed visual inspections of the tendon ends, bearing plates and surrounding concrete for those tendons affected by the containment opening Engineering Change (EC). These inspections were not required IAW IWL .

Rev. 1: The SGR-QC examination reports (File: Tendon Bearing Plate and Concrete Inspections.pdf) has been provided in this NRC folder.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/30/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 26

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Louis Lake

Category: Question

Request: Will PGN be doing the overall IWL inspection this R16 outage concurrent with ILRT?

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 10/26/2009

Response:

The IWL Inspections required by ASME Section XI are required every 5 years. CR3 last performed this inspection in R15 (2007). During R16 the ASME Section XI Repair / Replacement requirements require that a Pre-Service ISI VT examination be performed on the containment opening repair area prior to, during and following the ILRT. In support of the containment root cause it has been requested that an Augmented IWL Visual Examination be performed on the containment between Buttresses 3 and 4. This Augmented area includes the tendon gallery and the vertical face of containment only.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/4/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 27

Individual Contacted: Garry Miller **Date Contacted:** 10/22/2009

Requestor/Inspector: George Thomas **Category:** Question

Request: What was technical analysis for decision to detension only specific tendons? Provide the analysis?

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:** 10/26/2009

Response:

Summary:
1.The requirement to restore the concrete prestress within and around the steam generator opening to approximately the levels that existed prior to construction so that the original design margins could be maintained. This resulted in the need to de-tension 30 vertical tendons and 35 horizontal tendons.
2.The requirement to use the containment shell to move steam generators in and out of containment. This analysis was based on de-tensioning only the 10 vertical tendons and the 17 horizontal tendons within the steam generator opening. The remaining tendons adjacent to the opening were required to meet design loading conditions. The controlling load case was loss of decay heat removal accident in combination with the applicable loads from the polar crane.
White paper
The purpose of this white paper is to document the engineering processes and subsequent decisions made in identifying which tendons to detension in and around the CR3 temporary access opening in support of Steam Generators Removal (SGR) activities.
Sargent & Lundy (S&L) created Finite Element Models (FEMs) of the containment shell that are summarized in Ref. 1, Section 6.0. These FEMs were created using the GTSTRUDL program through the generation of a 3-D model of the containment which includes the containment shell, dome, basemat, representative soil springs and the equipment hatch. Similar to the design basis analysis, the models utilize thin shell elements that take into account bending and membrane action in the shell. Linear soil springs were also modeled similar to the design basis analysis to simulate the support provided by the rock foundation.
A significant goal of the SGR project team was to restore the prestress within and around the access opening to the design basis level prior to SGR and thus maintain the original design margins. S&L performed preliminary studies utilizing these FEMs to determine the optimum number of vertical and hoop tendons to detension outside the opening. These preliminary studies indicated that restoration of the prestress within the access opening was not possible unless the axial stiffness of the concrete sections within the access opening are nearly the same as or higher than the axial stiffness of the existing concrete sections around the opening. Ref. 2, Section 4.1 evaluated the mechanical properties of the new concrete in the opening and the existing concrete around the opening, including the effects of concrete creep (the time dependent increase in strain in the hardened concrete when subjected to sustained load, i.e. prestress). This evaluation was based on the requirements of ACI 209R-92, Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures, and was partially developed by Professor Domingo Carreira, Chairman of the sub-committee that prepared ACI 209R-92. This evaluation resulted in the requirement to add two layers of #11 at 11" centers to approximately equalize the stiffness of the concrete sections within the access opening with the axial stiffness of the existing concrete sections around the opening.
As part of the design evolution process, several detensioning alternatives were considered. This was necessary to meet the design constraint of keeping the number of tendons that have to be detensioned during the SGR outage construction to a minimum to minimize the duration of construction yet at the same time ensure that when the tendons are retentioned after the SGR opening has been plugged, the prestress within and around the

access opening is restored to approximately the levels that existed prior to SGR construction thus maintaining the original design margins.

Determination of Prestress Reduction Level

The analysis in Reference 2 to determine the number of hoop and vertical tendons to detension was performed using the following FEM models:

FEM Model A:

•Original design basis prestress. Tendon forces are based on original lock-off stress (0.7xFu) and losses at end of outage and EOL.

•Access opening has not been created yet and is not in model.

•Creep adjusted E is based on age of concrete when initially loaded and load duration to end of outage and EOL, i.e. E = 2681.62 ksi (for both end of outage and EOL)

•Element forces and stresses analyzed at end of SGR outage and EOL

FEM Model B:

•Same as Model #A except access opening is included in model.

•Tendon forces are based on original lock-off stress (0.7xFu) and losses at end of outage and EOL.

•Creep adjusted E is based on age of concrete when initially loaded and load duration to end of outage and EOL, i.e. E = 2681.62 ksi (for both end of outage and EOL)

•Reduced prestress in containment shell from de-tensioning 30 vertical and 35 hoop tendons is derived from two main load cases:

B1. All vertical and hoop tendons included in load case (as in Model A).

B2. Only the 30 vertical and 35 hoop tendons included in second load case

B3. Final reduced prestress = B1-B2

FEM Model C:

•This model reflects re-tensioning of the tendons at the end of the SGR outage. Young's modulus is the same for the new patch concrete and existing concrete, E= 3767.168 ksi (reflects the stiffening of the concrete section within the opening by adding rebar (#11s at 11" c/c vertical and horizontal, both faces).

•Include 30 vertical tendons and 35 hoop tendons. Tendon forces are based on re-tensioning to 0.7 Fu minus tendon losses to end of life (EOL).

By adding the results from Models B and C (at end of SGR outage and EOL) and comparing to the design basis results for vertical and hoop prestress from Model A , it can be determined if the prestress in and around the access and hatch area can be restored to pre-outage levels. The calculation (Ref. 2) determined that the prestress levels in and around the opening after re-tensioning would be at levels similar to those before the SGR outage.

Note: After Ref. 2 was issued it was decided by S&L and Progress Energy that since the creep adjusted Young's Modulus (E) of the new and old concrete have been equalized (by adding #11 rebar's to the access opening), that for all future analysis a reduced E value = 2500 ksi (Original design basis calculations were performed using a reduced Young's Modulus E=2500 ksi) would be used for both short and long term loads (Refer to Ref. 1, Section 6.0, Task 2 and Ref. 2, Attachment 5, pages 7 and 8 for further discussion concerning the use of E=2500 ksi).

Shell Analysis with Reduced Prestress for Activities Occurring During SGR

Based on the results of Ref. 2, i.e. detension a total of 35 hoops (17 in the opening and 9 above and 9 below the access opening) and 30 vertical tendons (10 within the access opening and 10 on either side of the access opening), S&L evaluated the containment shell (Ref.3) for activities occurring during the SGR as follows:

1.Modes 5 and 6 with the access opening created and the exposed liner plate in-place. The maximum number of tendons that may be detensioned should be such that no overstressing of the concrete shell or liner plate occurs for all accident load cases/combinations, including a LODHR accident.

2.Defueled (No Mode) with the access opening created in the concrete shell and liner plate for all construction loads resulting from rigging the steam generators (SGs) into and out of containment and for moving the auxiliary crane on the hatch transfer system (HTS). The maximum number of tendons that may be detensioned should be such that no overstressing of the concrete shell occurs.

Preliminary studies performed by S&L in the development of this calculation (Ref. 3) initially considered all 30 vertical and 35 hoop tendons detensioned prior to creating the opening, however, these preliminary studies revealed that the containment shell was grossly overstressed in this configuration when evaluated for loads resulting from moving the old SGs out and the new SGs into containment (these preliminary studies are not available). S&L determined that the maximum number of tendons that could be detensioned while lifting the SGs on the HTS is 10 vertical and 17 hoop tendons within the access opening. The remaining 20 vertical and 18 hoop tendons outside and adjacent to the access opening must remain fully tensioned until all lifting activities involving the SGs on the HTS are completed. The remaining 20 vertical and 18 hoop tendons outside and adjacent to the access opening may then be detensioned. This conclusion resulted in the containment shell having two stages of prestress:

Stage 1 Prestress - Reduced prestress based on de-tensioning 17 hoop and 10 vertical tendons within the opening. Applicable during Modes 5 and 6 descending and No Mode while the SGs are being moved on the HTS

Stage 2 Prestress – Reduced prestress based on de-tensioning 17 hoop and 10 vertical tendons within the opening and de-tensioning an additional 9 hoops above and below the opening (total of 35 hoops de-tensioned) and 10 additional vertical tendons on either side of the opening (total of 30 verticals de-tensioned). Applicable after all lifts involving the SGs on the HTS are completed thru Modes 6 and 5 ascending (Refueling).

The containment shell was evaluated in Ref. 3 for Stage 1 prestress, SGR opening with concrete removed but the liner plate intact and loads applicable during Modes 5 and 6 descending. Ref. 3 also evaluated the containment shell for Stage 1 prestress, SGR opening with concrete and liner plate removed, reactor defueled, and applicable loads for moving the SGs in and out of the containment. Ref. 5 evaluated the containment shell for Stage 2 prestress during Modes 6 and 5 ascending, prior to restoration of the opening, during which time a LODHR accident is the controlling load case in combination with the applicable loads from the polar crane. These Ref. 3 and 5 evaluations show that containment shell stresses for Stage 1 and Stage 2 prestress and the applicable loadings during the SGR construction sequence are within code allowables. The containment shell with all detensioned tendons retentioned, SGR opening plugged with concrete, and the liner plate opening welded back was evaluated for design basis loading (Ref.4) to show that the containment concrete and liner plate stresses are within code allowable and the as repaired containment has approximately the same design margins as the as-found containment prior to the SGR construction.

PrestressMODEConcreteLinerFuelHTSPolar CrnDB LoadsRef
Stage 15 and 6CutUncutOldNoYes (*)Yes (**)3
Stage 1No ModeCutCutNoneYes?No3
Stage 25 and 6CutRestoredNewNoYesNo5
AllAll PluggedRestoredNewNoYesYes4

(*) Dead weight of polar crane only
(**) Included design basis load combinations but substituted accident pressure and temperature resulting from a LODHR accident for LOCA pressure and temperature.

Restored Condition Analysis
The evaluations in Ref. 4 were performed using the following Finite Element Method (FEM) models:
FEM Model A:
•Original design basis prestress. Tendon forces are based on original lock-off stress (0.7xFu) and losses at end of outage and end of life (EOL).
•Access opening has not been created yet and is not in the model.
•E=2500 ksi for concrete (same as design basis calculations)
•Element forces and stresses analyzed at end of SGR outage and EOL

FEM Model B:
•Same as Model A except access opening is included in the model.
•Tendon forces are based on original lock-off stress (0.7xFu) and losses at end of outage and EOL.

•E=2500 ksi for concrete (same as design basis calculations)
•Reduced prestress in containment shell from de-tensioning 30 vertical and 35 hoop tendons is derived from two main load cases:
B1. All vertical and hoop tendons included in load case (as in Model A).
B2. Only the 30 vertical and 35 hoop tendons included in second load case
B3. Final reduced prestress = B1-B2
•Dead load of wet concrete within the opening is included
FEM Model C:
• This model reflects re-tensioning of the tendons at the end of the SGR outage. Young's modulus (E=2500 ksi) is the same for the new patch concrete and existing concrete (reflects the stiffening of the concrete section within the opening by adding rebar (#11s at 11" c/c vertical and horizontal, both faces).
•Include 30 vertical tendons and 35 hoop tendons. Tendon forces are based on re-tensioning to 0.7 Fu minus tendon losses to end of life (EOL)

By adding the results from Models B and C (at end of SGR outage and EOL) and comparing to the design basis results for vertical and hoop prestress from Model A , it can be determined if the prestress in and around the access opening and hatch area can be restored to pre-outage levels. The calculation determined that the prestress levels in and around the opening after re-tensioning would be at levels approximately the same as those before the SGR outage for a majority of the elements. Pre-outage prestress levels could not be restored to certain elements in and around the access opening. For these elements detailed stress evaluations were performed that demonstrated they met all applicable design basis allowable stresses.

References:
1.Calculation S06-0002, Revision 1, Containment Shell Analysis for Steam Generator Replacement – Design Criteria.
2.Calculation S06-0004, Revision 0, 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.
3.Calculation S06-0005, Revision 1, Containment Shell Analysis for Steam Generator Replacement – Shell Evaluation during Replacement Activities.
4.Calculation S06-0006, Revision 1, "Containment Shell Analysis for Steam Generator Replacement - Evaluation of Restored Shell"
5.Calculation S09-0025, Revision 0, Containment Shell Analysis for Steam Generator Replacement – Evaluation for Refueling prior to Restoration of Access Opening.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 28

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: What were forces acting on SGR opening area and adjacent areas:
A) Prior to tendon de-tensioning and concrete removal?
B) After de-tensioning and tendon removal?
C) After detention and concrete removal?

By this question, the NRC is seeking information to understand the structural behavior and response of the Containment Wall under real loads (i.e., Dead + applicable Prestress Load) in and around the SGR construction opening area for the configurations prior to, during and following creation of the SGR construction opening. Provide the pertinent information in an easily reviewable form.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector: 10/26/2009

Response:

Refer to Calculation S09-0048 stress plots. These plots are for dead load + vertical and hoop prestress as requested by George Thomas.
References:
1.Calculation S09-0048, Revision 1, Stress Plots for SGR Containment Analysis

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 29

Individual Contacted: Garry Miller **Date Contacted:** 10/22/2009

Requestor/Inspector: George Thomas **Category:** Question

Request: How were the forces acting on the buttress analyzed when the horizontal tendons were released and the forces became unbalanced?

Follow-Up:
From review of pages 90 thru 95 of Calculation S06-0005, "Containment Shell Analysis for SGR – Shell Evaluation During Replacement Activities," it appears that the unbalanced moment applied at the applicable buttresses as a result of detensioning of hoop tendons for creation of the SGR construction opening has been under-estimated. Based on the arrangement/spacing of hoop tendons and the basis for the value of Fex_hoop = 832.7 k/ft used in page 90 of the calculation, the unbalanced moment is expected to be higher than what is indicated on page 91 of the calculation. Please confirm/clarify if an error(s) was made in the referenced calculation and, if so, address its impact on the conclusions of the calculation S06-005 and other related calculations (if any) for the creation and restoration of the SGR construction opening based on EC 63016.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:** 10/26/2009

Response:

The unbalanced force(s) and moments from detensioning hoop tendons were evaluated for Buttress numbers 2, 3, 4 and 5 (Ref. 1, Pages 90 thru 95) and these forces and moments were applied to the appropriate nodes along the centerline of each buttress. Note that the forces and moments shown on pages 90 thru 95 of Ref. 1 are in the direction of the tensioned tendon. When these tendons are detensioned the signs reverse (Ref. 1, Attachment 2, load cases 6 and 10 and load combinations 102 and 104). The unbalanced forces are derived from the original lock-off stress – tendon losses at the time of the steam generator replacement outage (Ref. 2, Section 4.2.1.2).
References:
1.Calculation S06-0005, Revision 1, Containment Shell Analysis for SGR – Shell Evaluation During Replacement Activities.
2.Calculation S06-0004, Revision 0, Containment Shell Analysis for SGR –Properties of new Concrete for Access Opening and Number of Hoop and Vertical Tendons to be Detensioned.

Follow-up Response (10/28/10):

The follow-up request for Question 29 regarding unbalanced moments at the buttress resulting from the eccentricity of the hoop tendon anchorages after the initial phase 1 detensioning was reviewed and it has been determined that an error was made in calculating the unbalanced moment. (Calculation S06-0005, Containment Shell Analysis for SGR- Shell Evaluation during Replacement Activities) The calculation for the out-of-balance force divided the unit stress by 3 rather than 2, thereby underestimating the unbalanced moment. NCR 429601 was initiated to document the error and revise calculation S06-0005.

The calculation in which the error was made (S06-0005) supported the original phase 1 SGR detensioning sequence. After the delamination occurred, this calculation (S06-0005) was replaced by calculations that supported detensioning the increased scope of horizontal and vertical tendons (S10-0004 Tendon Detensioning Calculation). This calculation has not only included the initial Phase 1 SGR detensioning scope but also the other horizontal and vertical tendons that were detensioned to facilitate the Bay 34 repair. A path dependent load history was maintained to ensure stress redistribution due to construction activities was accounted for in the final design basis calculations.

If the increase in the unbalanced moment would have cause damage to the concrete, the most likely result would have been vertical cracking in the circumferential panels between the SGR opening and Buttresses 3 and 4, not in the buttresses themselves. These areas were visually inspected by the SGR project responsible engineers because the concrete in bay 34 was included in the analysis to support movement of the steam generators. The inspections did not identify any vertical cracking in these areas.

These inspections validate that underestimating the unbalanced moment in S06-0005 did not impact the conclusions of S06-0005 regarding potential damage to containment at the buttresses. As discussed above, S06-0005 will be revised to annotate the error that was made.

Reference for Response to Follow-up Request

1. S10-0004, Tendon Detensioning Analysis

Misc Notes:

Response By:	Don Dykshterhouse		
Reviewed By:		**Date Response Provided:**	10/28/2010
Status:	Closed	**Date Closed:**	10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 30

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: Where is PII based, and provide a description of their credentials?
A) What is their root cause approach?

Provide PII's failure mode chart referred to in item (5) under the title, "Unique Qualification" of the response.

Identify the root cause failure analysis report for the MOX facility referred to in Item (6) under the title "Unique Qualification" of the response, if submitted to the NRC, or provide a copy of the report.

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

PII location, background, qualification and methods were reviewed with George Thomas. A hard copy of the response was provided and discussed on 10/28/09. Electronic copy of this file is in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 30, Q24 Response Info - Williams

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 10/28/2009

Status: Closed

Date Closed: 5/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 31

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: What are the various root causes and fault tree scenarios being considered?

Provide a list of root cause failure modes being considered under each of the 9 broad categories (i.e., break down each of the 9 categories into the approximately 79 failure modes being evaluated for CR3 containment).

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

Potential causes being considered.
The Root cause team has initially identified nine categories or groupings of potential causes. They are shown below. Under each category, there are numerous potential causes that have been identified and are being addressed through a support/refute process. The potential causes and support refute process are in progress and not yet finalized.

1. Concrete Design – ie analysis of radial loads, influence of equipment hatch opening
2. Concrete Construction – ie concrete placing or vibration
3. Concrete Materials – ie weak aggregate, concrete slump
4. Concrete Shrinkage, Creep, and Settlement – ie creep, shrinkage
5. Chemical or Environmental Aging – ie corrosion, grease
6. Concrete-Tendon-Liner Interaction – ie tendon relaxation, load distribution
7. Concrete Removal Processes – ie hydro-demolition, de-tensioning process
8. Operational Events – ie containment spray, ILRTs
9. External Events – ie weather, seismic

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 10/28/2009

Status: Closed

Date Closed: 2/3/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 32

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: When and what will be the deliverable for the NRC to review, i.e., schedule for root cause, NDE, results of core bore samples, and design basis analysis?

Provide a response to part of the original question “What deliverables related to root cause analysis, extent of condition (NDE/core bores), design basis analysis and repair options would be provided to the NRC for review?”

Provide weekly updates to the schedule.

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

I asked George Thomas for a clarification of this request on 10/28/09. He said he would like a copy of the current schedule for activities for the Root Cause, Condition Assessment, Design Basis and Repair teams. A hard copy was provided on 10/29/09. Electronic copy of this file is in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 32, Q26 Response Info - Williams

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 10/28/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 33

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Provide copy of PGN's and PII's Root Cause Analysis procedure.

Include a statement on PII's root cause analysis procedure or if they would be working to PE's procedure.

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 10/26/2009

Response:

A hard copy of the PGN root cause procedure CAP-NGGC-0205 was provided to George Thomas on 10/28/09. PII does not have a written procedure. The PII Root Cause process was discussed with George Thomas as part of response to Request 30. Electronic copy of this file is in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 33, Q27 Response Info - Williams

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 10/28/2009

Status: Closed

Date Closed: 2/25/2010

20-Apr-11 7:56:35 AM

Request Number: 34

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: Who is the contractor doing Design Basis Analysis? How does this relate to Root Cause analysis efforts?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector: 10/26/2009

Response:

The selected vendor to perform Design Basis Analysis is MPR Associates, Inc. Alexandria, Virginia. Computer Aided Engineering (CAE) Associates, Middlebury, Connecticut, is supporting MPR in the development of the 3-D finite element model.

The Root Cause Analysis team efforts are being supported by Performance Improvement International, PII, Oceanside, California and has independent technical capabilities to support the Root Cause Analysis team. The root cause(s) identified by the Root Cause Analysis team will be evaluated by the Design Basis Analysis team for impact on the design analysis and on the design basis.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 35

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Robert Carrion

Category: Question

Request: Are you changing the design or licensing basis? Will a License Amendment or 10CFR50.59 type analyses be required?
A) Are you changing the ACI 318-63 code of record?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector: 10/26/2009

Response:

The intent of the design analysis and the engineering change to implement the repair is not to change either the design or licensing basis? A 50.59 evaluation will be performed when the engineering change is being developed. It is the intent of the design analysis and the engineering change not to change the code of record ACI 318-63.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Open

Date Closed:

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 36

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Robert Carrion

Category: Question

Request: Will there be a past-operability analysis completed?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector: 10/26/2009

Response:

A determination of past operability is not required. A reportability evaluation was made in NCR 358724-22. The conclusion of the reportability evaluation was that the delamination occurred during the creation of the steam generator opening in September, 2010. Based on this, the delamination occurred in Mode 5, after CR3 was shut down for R16. Per Technical Specification 3.6.1, containment is required to be operable in Modes 1-4. Therefore, the condition is not reportable and a past operability evaluation is not required.
References: NCR 3578724 assignment 22

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 7/21/2010

20-Apr-11 7:56:35 AM

Request Number: 37

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: George Thomas

Category: Question

Request: What type of analysis and codes (by names) is expected to be used in the design basis analysis?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector: 10/26/2009

Response:

The analysis computer code that will be used for the design basis analyses is ANSYS Version 11.0 SP1.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	38		
Individual Contacted:	Garry Miller	**Date Contacted:**	10/22/2009
Requestor/Inspector:	Anthony Masters	**Category:**	Information Request

Request: Provide procedures and drawings for tendon installation and stressing in original construction (containment walls and dome), and also after the 1976 dome repair.

References:

Response Assigned to:	Don Dyksterhouse	**Date Due to Inspector:**	10/26/2009

Response:

Design drawings for both original design and post-dome repair are included in the CR3 Document Control System. Generally the drawing series that start with 421-001 is the original plant design drawings. The series that starts with 421-300 contains the dome repair drawings. Specifications for concrete and reinforcement are included in the shared drive. Drawing copies are included in the drive where available. Several of the 421-300 series of drawings are available only on aperture cards. A drawing list is in the Excel file.

L:\Shared\CR3 Containment\ROOT CAUSE ANALYSIS Files\(1) Concrete Design\Concrete Design Drawings

Misc Notes:

Response By:	Glenn Pugh		
Reviewed By:	Charles Williams	**Date Response Provided:**	10/28/2009
Status:	Closed	**Date Closed:**	7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 39

Individual Contacted: Garry Miller

Date Contacted: 10/22/2009

Requestor/Inspector: Louis Lake

Category: Question

Request: Were there any changes to the dome made in 1976 (additional new anchors and/or radial rebars)?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector: 10/26/2009

Response:

The referenced report and drawings indicate radial #6 reinforcing bars were added and # 11 bars were used to replace damaged # 8 circumferential bars. There were approximately 1,850 radial #6 reinforcing bars added. If any #8 circumferential bars were damaged during concrete removal and the entire hoop was to be replaced, a #11 bar was used in place of the #8 bar. If any #8 circumferential bars were damaged during concrete removal and only a portion of the bar was exposed, a new # 8 bar was cadwelded to the embed bar.

References:

Final Report - Reactor Building Dome Delamination Report, December 10, 1976

SC-421-341, Reactor Building – Concrete Dome Repair Dome Reinforcement North Half – Top Reinforcement

SC-421-342, Reactor Building – Concrete Dome Repair Dome Reinforcement South Half – Top Reinforcement

SC-421-343, Reactor Building – Concrete Dome Repair Dome Reinforcement North Half – Bottom Reinforcement

SC-421-344, Reactor Building – Concrete Dome Repair Dome Reinforcement South Half – Bottom Reinforcement

SC-421-345, Reactor Building – Concrete Dome Repair Dome Reinforcement Sections & Details

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 11/13/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	40		
Individual Contacted:	Garry Miller	**Date Contacted:**	10/22/2009
Requestor/Inspector:	Anthony Masters	**Category:**	Question

Request: What is the cause of the low spot on the dome?
A) Email from Lese said it was same as previous inspections since 1976. Can this be confirmed from the final documentation and photographs in 1976?

References:

Response Assigned to:	Don Dyksterhouse	**Date Due to Inspector:**	10/26/2009

Response:

The construction microfiche database contains a listing of microfiche for the dome repair project. The cards range in number from 2C01024 to 2C02089. A search of the database titles showed several microfiche cards (2C02064 and 2C02065) containing nonconformance's and corrective actions for the repair project. A review of these microfiche records did not reveal any information on a low spot. A check of the pour cards also did not mention a low spot or other problem.

However, to help in answering this question a conversation was held with Mr. Earnest Gallion about this repair. Mr. Gallion was an employee at the time of the dome repair. He reported that the concrete finishers used at the time of the repair where not as experienced as could be. There were several low spots and other imperfections that existed from the initial concrete pours. These are not considered detrimental to the qualification of the dome. Would also consider that these existing since the repair project.

This confirms statements by Mr. Joe Lese.

A copy of the Construction Microfiche log is included here: L:\Shared\CR3 Containment\ROOT CAUSE ANALYSIS Files\(2) Concrete Construction\Construction MicroFiche Index.pdf

Misc Notes: Related to question #1

Response By:	Glenn Pugh		
Reviewed By:	Charles Williams	**Date Response Provided:**	10/28/2009
Status:	Closed	**Date Closed:**	2/25/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	41		
Individual Contacted:	Garry Miller	**Date Contacted:**	10/22/2009
Requestor/Inspector:	Anthony Masters	**Category:**	Question

Request: NCR 360269 mentions SGR expected flexible tendon sheaths? What was the basis for them expecting a thin wall sheath?

References:

Response Assigned to:	Charles Williams	**Date Due to Inspector:**	10/26/2009

Response:

Enclosed in this folder in response to the above question:
FW_ NRC Question - D Jopling Response.pdf

Misc Notes:

Response By:	Rick Portmann		
Reviewed By:		**Date Response Provided:**	11/2/2009
Status:	Closed	**Date Closed:**	1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	42		
Individual Contacted:	Garry Miller	**Date Contacted:**	10/22/2009
Requestor/Inspector:	Anthony Masters	**Category:**	Question

Request: Were radial tension stresses due to the hoop tendons considered in the original design?

References:

Response Assigned to:	Don Dyksterhouse	**Date Due to Inspector:**	10/26/2009

Response:

Cannot readily determine from the old Gilbert Calculations what the direct answer is to the request. It appears that the tendon design is based on limiting the concrete tensile stress to 212 psi. This limit bounds the tensile stresses in meridional, and hoop directions. See Book 2, Section 1.01.7, pages 1.01.7/6 and 1.01.7/7 for a brief memorandum outlining the critical loading of the cylindrical RB wall. The tendon pre-stress is designed to limit the tensile stresses in the concrete for the load combinations. However, it does not appear that the calculations considered the tensile stresses in the concrete outside the tendon's influence.

Copies of calculation pages are included at following drive location:

L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 42, Q36 Response Info- Pugh

Misc Notes: Consideration is on-going by George/Anthony

Response By:	Glenn Pugh		
Reviewed By:	Charles Williams	**Date Response Provided:**	10/28/2009
Status:	Closed	**Date Closed:**	1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 43

Individual Contacted: Dennis Herrin

Date Contacted: 11/2/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: Please provide Drawings: SC-400-007, 008, 009, and 015; and S-425-011 and S-425-012
Specifications: SP-5566, 5569, 5583, 5618, 5648, and 5909
Reports: VT-3C Report VT-07-106 and VT-3C Report VT-07-111
Calculations: S-07-0019 and S-07-0033

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector: 11/2/2009

Response:

Enclosed in this folder in response to the above question: All requested information provided except for SP-5566, SP-5583. 11/3/09 Update. The last 2 spec's requested have been included in the file.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 44

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: In continuing evaluation of the IWL inspection and maintenance program:

If possible, I would like to speak with Mr. Bernard Komara and Mr. Marc LeBlanc as they were listed as the inspectors on two previous inspection reports that I have reviewed.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Mr. Marc LeBlanc (NIC Contractor Inspector) was here in refuel 15 (2007) and to my knowledge is not here for refuel 16. Mr. Bernard Komara (NIC Contractor Inspector)has returned to CR3 for refuel 16 and is working for the site QC Organization. The Supervisor for Mr. Komara is Jeff Bennett.. Please contact Jeff (x-3323) for Bernie's availability.

Misc Notes: Evaluation of containment liner bulges still in progress.

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 45

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, indicates compliance with the 1992 addenda of the 1992 Edition of ASME Section XI, Subsection IWL, while the document titled ASME Section XI/ASME OM Code Program, Interval 4: Containment Inspection Program (2nd CISI) Revision 3 (Dated 5/6/09) indicates the 2001 Edition through the 2003 Addenda. Please clarify.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The last performance of the Tendon Surveillance under SP-182 was in 2007. The ASME Section XI code of record during that time was the 1992 addenda of the 1992 Edition of ASME Section XI, Subsection IWL. In accordance with 10CFR50.55a, licensees are required to update their ISI Programs to meet the requirements of ASME Section XI once every 10 years or inspection interval. The 3rd inspection interval was completed on August 13, 2008 and the new interval (4th) began on August 14, 2008. For the 4th interval , the 2001 Edition through the 2003 Addenda is the code of record. The SP-182 will be revised to reflect the new code edition prior to its next required 5 year tendon surveillance.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 46

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, has some concrete inspection activities associated with it as part of the tendon surveillances. Are the documented and reported in separate documentation or are the VT-1C and VT-3C examinations credited for this (i.e. VT-07-111 and VT-07-289)? If not, I would like to review the additional documentation.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The visual examinations for the tendon surveillances are documented separately from the IWL concrete examinations. The last two tendon surveillances and the last two IWL examination reports have been supplied. See the Request #24, NRC SIT Question #18 folder for these examination reports.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 47

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program,, Section 3.5.3.1 specifies requirements for calibration for all measuring devices. I would like to review a sample of those records also.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The tendon surveillance reports have the calibration records for the tendon testing equipment. The last two tendon surveillances reports have been supplied. See the Request #24, NRC SIT Question #18 folder for these examination reports.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 48

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, Section 3.6 specifies acceptance criteria. Section 3.6.2 states that "abnormal conditions determined as the result of a visual inspection of the exterior concrete surface of the containment shall be recorded and documented, and investigated by Engineering for possible degradation of the structure."
Also, "Cracks found in concrete adjacent to the tendons (within 2 feet of the bearing plate) having widths greater than 0.010 inch shall be recorded and reported to Engineering for evaluation and resolution. Any crack widths greater than 0.050 inch shall be cause for investigation by Engineering to determine the cause and if there is any abnormal degradation of the structural integrity of the containment."
Photographs VT-07-289-8 and VT-07-289-11, which are associated with VT-1C Report VT-07-289, appear to show cracks within 2 feet of the bearing plate. Have these been documented and evaluated?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The SP-182 criteria specified applies to the anchorage and bearing plate inspections performed for the tendon surveillances. The reports discussed are from the ASME Section XI IWL examinations performed. The recording and acceptance criteria may differ as the performance requirements come from separate requirements. These particular indications described on R15 IWL Report VT-07-289 were included in NCR 256010 for evaluation.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 49

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, Section 3.7.1 recommends equipment for implementation of this inspection and 3.7.1.12 lists "optical comparators with 0.005 inch accuracy for measuring crack widths in concrete." Is this being used? VT-07-111 and VT-07-289 do not have it listed in the inspection equipment area on the reports. These reports list a 6"scale and measuring tape. Is 0.005 inch accuracy (or the 0.010 inch as acceptance criteria section 3.6.2 states) possible with these?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The inspection reports referenced were performed as part of the IWL Examinations. The controlling procedures are NDEP-0620 and NAP-02. The SP-182 surveillance procedure referenced is used in conjunction with the Tendon examinations (not the IWL Examinations). The accuracy stated comes from the PSC Procedures and equipment utilized for the Tendon Examinations. An example of the certification record for one of the past surveillances can be found on pages 77-78 of the 6th surveillance report {WR 341602_ 6th-Surv.pdf}. Copies of the certifications have been enclosed in this file. This report can be found:
L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 24, Q18 Response Info-Portmann\IWL - Tendon Surveillance History

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/12/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 50

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, Section 3.7.2.11 states as an prerequisite to "verify that stressing jacks, pressure gauges, comparators, and all other measuring devices have been calibrated per Step 3.5.3.1…" Are the measuring devices used calibrated per Step. 3.5.3.1?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Measuring devices are calibrated per Step 3.5.3.1 of SP-182. An example of the certification records for one of the past surveillances can be found on pages 58-82 in the 6th surveillance report {WR 341602_ 6th-Surv.pdf}. This report can be found:
L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 24, Q18 Response Info-Portmann\IWL - Tendon Surveillance History

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/12/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 51

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, Enclosure 1 lists tendons in the 5th and 7th surveillance as 46H21, 46H28, etc…; however, Enclosure 11 indicates that they are numbered as 64H21, 64H28, etc… I believe these are in fact the same tendons, but should the numbers not be consistent?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

These are the same tendons. The first two digits of the horizontal tendon identification refer to the tendon series on the containment buttresses it spans (ie. Between buttresses 4 and 6 [46Hxx] is the same as between buttresses 6 and 4[64Hxx]). Over the years CR3 has not been consistent in the use of one versus the other. A spreadsheet has been provided showing the tendon identifications used over prior surveillances. [Note: the spreadsheet is not a controlled document, just an aid for review of previous surveillance documentation.]

Enclosed in the Request# 51 folder:
Spreadsheet: Tendon Identification History (#51).xls

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 52

Individual Contacted: Dennis Herrin

Date Contacted: 11/3/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, Enclosure 5 is titled "Reduced Force Dome Tendons" and lists 18 tendons. What is meant by this term "reduced force"? When, how, and why did they become reduced? D 125 is shown on this list and is also listed as tested in the 3rd Surveillance. Please clarify.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Following the investigation and evaluation of the 1976 Dome delamination event the dome tendons were re-stressed to predetermined values, of which approximately every 8th tendon was stressed at a value much, much lower than the remaining tendons (Approx. 646 KIPS vs. 1635 KIPS). These tendons are exempt from tendon lift-off, and wire removal testing.
During the random selection process if one of these exempt tendons (or in general a tendon that is inaccessible or due to interferences cannot be safely tested per the IWL code) happens to be selected for testing, then a substitute tendon located as close as possible to the exempt tendon gets selected for examination and testing.
Although still classified as exempt, the original exempt tendon is still subject to the examination tendon anchorage, free water and corrosion protection medium examination requirements if possible.
A review of the 3rd Surveillance tendon lift-off data shows that tendon D123 was tested. No test data was found for D125.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/18/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 53

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

SP-182, Rev. 16 (Dated 5/22/09) Reactor Building Structural Integrity Tendon Surveillance Program, Enclosure 11 lists original lift-off values. Are the values for the dome in this listing before or after the repair?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The values listed in SP-182, Enclosure 11 are following the 1976 delamination event repair of the Dome.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 54

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: In continuing evaluation of the IWL inspection and maintenance program:

I have reviewed some inspection reports for the IWL inspections for the shell, but would like to review some reports and evaluations for inspections on the dome.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The last two IWL examination reports for 2001 (R12 and 2007 (R15) have been supplied and include examination of the dome. See the Request #24, NRC SIT Question #18 folder for these examination reports.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 55

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

VT-07-111 and VT-07-289 documents some cracks and spalls and measured depths. How were the depths obtained for the cracks and spalls?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Direct Visual Examination was conducted on RBCN-0015 during R15 using the suspended work platform, a man lift (around the equipment hatch), and a step ladder (lower elevations not accessible by suspended work platform or man lift).
Using the procedure and criteria provided in the Engineering letter as threshold for recording, the VT-3C was performed and any areas of distress identified were further evaluated during a VT-1C. The VT-3C also considered areas of distress not previously identified, as well as changes to previously identified areas of distress.
During the VT-1C, previously existing areas of distress were compared with previous data and further characterized to document changes to previous data recorded. Areas of distress not previously identified were characterized and recorded. In all cases, size and depth were dimensioned and recorded with a tape measure and 6” scale. A short length of 3/32” bare wire welding rod was used for tight spots where the 6” scale would not fit. Technique used with the bare wire was to insert into the opening, and measure maximum depth against the 6” scale.

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/18/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 56

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

VT-07-111 and 289, Item #11 indicates that spalls were due to some embedded cables near boxes (shown in photographs VT-07-289-6 and VT-07-289-15). What were these cables?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

It is believed that these cables and boxes are abandoned remnants from the testing equipment utilized during the original Structural Integrity Test in 1976. (stress & strain gages etc.)

Misc Notes:

Response By: Rick Portmann

Reviewed By:

Date Response Provided: 11/9/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 57

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

The names and dates on theVT-1C and VT-3C reports are identical. Are both inspections/reports done on the same day by the same staff?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Yes, the VT examiners know that certain indications found during a VT-3 examination require an additional, closer VT-1 examination and may elect to perform both examinations in series since they are already at the area.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 58

Individual Contacted: Dennis Herrin **Date Contacted:** 11/4/2009

Requestor/Inspector: Anthony Masters **Category:** Question

Request: In continuing evaluation of the IWL inspection and maintenance program:

FSAR, Section 5.2, Section 5.2.5.2.1.1.h.5 states: 5. The surveillance was performed 1, 3, and 5 years after the initial containment structural integrity test and is performed every 5 years thereafter. A report of each inspection will be recorded and significant deterioration or abnormal behavior reported to the Commission.
Are significant deterioration or abnormal behaviors being reported to the Commission?

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

Yes - Significant deterioration or abnormal behaviors are being reported to the Commission. SP-182 (Para's 5.3.2 and 5.3.4) and the Improved Technical Specifications (5.7.2 Special Reports) describe the reporting requirements.

Misc Notes:

Response By: Rick Portmann

Reviewed By: **Date Response Provided:** 11/12/2009

Status: Closed **Date Closed:** 2/25/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 59

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: In continuing evaluation of the IWL inspection and maintenance program:

I would like to review SP-180 and understand the basis surrounding the use and discontinuance for inspections of the dome repairs.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

A copy of SP-180 for inspection of the dome was provided to George Thomas on November 18, 2009. The document was obtained from microfiche and is not available electronically. Also note that the procedure was developed and implemented prior to the development of the IWL program.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 2/25/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 60

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Louis Lake

Category: Question

Request: What are the repair options being considered as a fix to the CR3 containment delamination issue?

References:

Response Assigned to: Sammy Radford

Date Due to Inspector: 11/6/2009

Response:

There were two options that had being considered.
1.Remove the delaminated concrete that is between is between Buttress #3 and Buttress #4 and install additional rebar ties. The wall will be reformed and replaced with new concrete. This was the method used to repair the delaminated dome section during construction and the method we will be using .
2.The next option we considered was to install anchors into the solid concrete portion of the wall on a spacing to be determined and anchor the delaminated section and solid section together. Then we will be pressuring grouting the delamination using a cementitious grout and epoxy grout to bond the two layer. We will be using some NDT to ensure we have filled all the voids between the two layers. This option was eliminated due to problems identified with the use of the grout with the potential of the debris blocking flow paths of the grout and size of some of the crack areas.

Misc Notes:

Response By: Sammy Radford

Reviewed By:

Date Response Provided: 11/18/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 61

Individual Contacted: Dennis Herrin

Date Contacted: 11/4/2009

Requestor/Inspector: Louis Lake

Category: Question

Request: What post modification testing of the CR3 containment is being planned to be performed following repair of the delaminated condition in order to demonstrate structural and leak-tight integrity?

Follow up Request:

10)This is a follow-up question based on response provided to Request #61 with regard to post-mod containment pressure testing. The response provided is incomplete and not adequate for the major repair of the containment delamination. Please provide a complete considered response supported by a technical basis.

References:

Response Assigned to: Paul Fagan

Date Due to Inspector: 11/6/2009

Response:

We are looking at the requirements for post mod testing. At the present time we plan to use the ILRT as the post mod testing.

Follow up Response:

CR3 plans to perform a structural integrity test at 1.15 x the containment design pressure in accordance with ASME Section III, Division 2, Article CC-6000. Radial and vertical displacements, as well as strain in the repaired wall, will be monitored and recorded during the test for comparison to predicted values. This test is intended to demonstrate the structural integrity of the containment building. An ILRT will be performed after the structural integrity test to demonstrate acceptable leakage performance. Surface examinations will be performed as required.

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 8/18/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	62		
Individual Contacted:	Charles Williams	**Date Contacted:**	11/18/2009
Requestor/Inspector:	George Thomas	**Category:**	Information Request

Request: Provide strain gage data and map.

References:

Response Assigned to:	Paul Fagan	**Date Due to Inspector:**	11/18/2009

Response:

Strain gage and displacement data provided on 11/18/09. Electronic copies available on L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\FAGAN Q-A\Request 62 - Worthington - Williams

Misc Notes:

Response By:	Worthington		
Reviewed By:	Charles Williams	**Date Response Provided:**	11/18/2009
Status:	Closed	**Date Closed:**	1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 63

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: Anthony Masters

Category: Information Request

Request: Provide survey data results for the dome [repeated survey surveillance test], internal diameter of containment and survey data results for external buttresses.

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #63

Misc Notes:

Response By: Martin Souther

Reviewed By:

Date Response Provided: 6/24/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 64

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: Dan Naus / George Thomas

Category: Information Request

Request: Provide evaluation of crack identified in AR 368389 [core number 54 below the equipment hatch].

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #64

Misc Notes:

Response By: Martin Souther

Reviewed By:

Date Response Provided: 6/24/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number:	65		
Individual Contacted:	Garry Miller	**Date Contacted:**	12/2/2009
Requestor/Inspector:	George Thomas	**Category:**	Information Request

Request: Provide credentials of MPR Associates and CAE , specifically with regard to concrete containment structural analysis and design for nuclear plants.

References:

Response Assigned to:	Don Dyksterhouse	**Date Due to Inspector:**	

Response:

The following data are excerpts from the MPR Associates response to a request for proposal RFP JO09-011. The responses are listed if the experience involved reinforced concrete analysis at a nuclear power plant if either MPR Associates or CAE were involved in the projects listed.

1.Development of the 3-D model for Three Mile Island nuclear power plant.
2.Original analysis of Crystal River containment dome delamination report.
3.Structural analysis of the reinforced concrete Fuel Handling Building at Salem Nuclear Plant.
4.Development of models for structural analysis of concrete containment buildings at Turkey Point and Oconee nuclear power plant.

MPR Associates has supported the nuclear industry since 1964.
CAE has supported the nuclear industry since 1993.

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	
Status:	Closed	**Date Closed:**	8/30/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:35 AM

Request Number: 66

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Please confirm that the condition assessment, design basis analysis, root cause analysis, and repair option analysis efforts, currently ongoing for CR3, account for the following: SGR construction sequence (initial tendon detensioning, concrete removal, additional tendon detensioning, concrete placement, repair, tendon retensioning) loading and stiffness, based on the extent of condition of the affected areas, and is properly considered to account for the stress redistribution in the containment wall within the opening and its adjacent areas.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The design analysis model has been generated using a path dependent loading history to determine the current state of stress in the containment structure. The events in this path dependent loading history were modeled as:

1.Existing structure prior to R16
2.Concrete removal
3.Removal of Containment Liner
4.Installation of Containment Liner
5.Tendon detensioning
6.Delamination

To evaluate the detensioned condition and the end-of-life condition, the detensioning sequence is being evaluated based on the scope and sequence that will be used. After detensioning, the design basis calculation will evaluate the containment structure using a path dependent loading that includes concrete placement followed by retensioning.

By modeling the structure as a path dependent loading, stress redistribution of loading such as dead load will be redistributed to the current condition of containment.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 5/10/2010

Request Number: 67

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Question

Request: Refer to Slide #59 of the 11/20 public meeting presentation. This is with regard to how the liner is modeled for the Design Basis Analysis. Based on your current design basis in the FSAR and Containment Design Basis document 1/1, the liner serves as a leak-tight membrane during operating and accident conditions, and not as a structural element resisting design basis loads. However, in your current FEA model developed for the delamination issue, the liner seems to be included as a structural load-carrying member.

Explain and justify how the way the liner is modeled in the ANSYS model are consistent with your current design basis?

How will the liner be evaluated against design basis acceptance criteria?

How will you evaluate the effects on the liner during detensioning, repair, and retensioning?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The initial response (Sections 1.0 thru 8.0 below) provided previously to the S.I.T. was partially based on hand calculations comparing the compressive stresses in the containment wall, with and without the liner plate included in the evaluation. The conclusion was that including the liner plate in the model resulted in lower compressive stresses in the concrete, and that the liner plate remains in compression for the critical load case of Dead Load + Prestress +1.5 Design Pressure. An additional check has now been made using the ANSYS finite element model (FEM) developed by MPR Associates to evaluate the effects of removing the liner plate from the containment shell model, for all design basis loads. The results are discussed in Supplement #1 to this question.

Note that the initial response below has been revised to incorporate S.I.T. comments.

Table of Contents:

Attachment 1:
Mathcad calculation comparing concrete and liner plate stresses for individual load cases and the design basis accident pressure load combination considering the liner plate both active and inactive in resisting applied loads.

Attachments 2 thru 4:
Pages copied from the original Gilbert Associates Design Basis Calculations for the containment that indicate the

liner was included in the original Kalnins model for evaluating the containment shell.

1.0 Introduction:
This response explains the logic behind the decision to include the liner plate in the containment ANSYS finite element model (FEM). The applicable design criteria for the liner plate are established based on the CR3 FSAR and Containment Design Basis Document (DBD). The following questions are addressed:

(i) What are the design basis acceptance criteria for the liner?
(ii) How was the liner plate modeled in the ANSYS FEM?
(iii) What was the justification for including the liner in the ANSYS FEM?
(iv) Was the liner plate considered in the strength evaluation of the containment wall?
(v) What is the effect on the liner plate from detensioning and retensioning tendons?

2.0 Background Information:
The CR3 prestressed concrete containment is provided with a steel liner plate pressure boundary thus ensuring leak tightness during testing, operating and post-accident conditions. The nominal liner plate thickness is 3/8” for the cylinder and dome and 1/4" for the base. The liner plate has vertical stiffener angles welded to it that are embedded in the concrete. The resulting composite section will follow the deformed shape of the containment thus ensuring strain compatibility between the steel liner and concrete containment shell under various operational and accident loads. The liner plate stiffeners are generally vertical steel angles 3”x 2” x ¼” spaced nominally at 18” center to center; there are no horizontal stiffeners. The liner plate is fabricated from ASTM A283 Grade C carbon steel with minimum yield strength of 30,000 psi. Stiffener angle spacing ensures that the critical buckling stress is greater than the proportional limit of the steel (Refer to Section 4.0 below for Design Basis Acceptance Limits).

3.0 Conclusions:
FSAR, Section 5.2 states that the liner and concrete act compositely, therefore, the liner plate should be included in the ANSYS finite element model to accurately represent the interaction between the liner and concrete in determining maximum strains and stresses. Including the liner in the model will result in a true representation of the stiffness and corresponding load distribution in the structure from composite action between the two materials.

Including the liner plate in the model is also conservative. The effect is less benefit to the concrete from prestressing. Vertical and hoop concrete compressive stresses for the critical load case of Dead Load + Prestress + 1.5 Design Pressure are lower when the liner plate is included in the model when compared to stresses resulting from a model that excludes the liner (Refer to Attachment 1).

The liner plate has been ignored in determining the steel reinforcement requirements for the containment shell strength evaluation. The strength of various sections through the containment wall (and dome) have been evaluated by both Working Stress Design and Ultimate Strength Design methods per the requirements of the FSAR and ACI 318-63. In the repair area, where the stress demand exceeded the sections capacity, additional steel reinforcement has been provided, ignoring the presence of the liner plate steel (Reference Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall).

The design basis limit for the liner in tension is the strain associated with the minimum yield strength. Other design requirements for the containment shell result in liner plate stresses that never go into tension for both normal operating and accident load conditions (verified by Calculation S10-0058, Evaluation of Vertical Cracks Outside the SGR Bay, and Attachment 1). The liner plate actually acts as a loading element during all normal and accident load scenarios. Note that ANSYS FEM analysis is performed to evaluate the effects of modifications in Bay 3-4 and identify any anomalies that may occur due to local discontinuities in and around the SGR access opening. Acceptance of the anomalous conditions is based on FSAR strain limits.

As a result of the SGR, concrete tensile stresses will exist in and around the repaired access opening that may exceed the FSAR acceptance criteria of $3\sqrt{f'c}$ and $6\sqrt{f'c}$ for membrane and membrane + bending. Since the liner is not generally considered as steel reinforcement in the strength evaluation of the concrete cross-section, additional reinforcement is added to the containment wall to accommodate these tensile forces.

4.0 Design Basis Acceptance Criteria:
As noted in Section 4.2-c the liner compression strain is limited to 0.005 in/in and the average tensile strain must not exceed that corresponding to the minimum yield stress, i.e. maximum average tensile strain = 30000/29E6 = 0.00103 inch/inch. FSAR Section 5.2.5.2.2 states that the liner has been designed so that the critical buckling stress is greater than the proportional limit (yield) of the steel.

The design basis of the CR3 containment preceded standards and codes which contained liner strength provisions, specifically, the ASME Code and its predecessor ACI 349-72 (Refer to Sections 4.4 and 4.5 for details). Instead, the design of the CR3 containment was based on ACI 318-63 which contained no liner plate strength provisions; consequently, the FSAR does not have any liner plate strength provisions either. A review of both the FSAR and DBD 11 (Containment Design Basis Document) indicates that there are no statements in either that prohibit using the liner as a strength element. As noted in Section 4.2-a, “The liner has been anchored to the concrete so as to ensure composite action with the concrete shell”. This statement suggests that the liner plate should be included in any analysis of the containment shell for load distribution. Additionally, the liner is included in the computer model described in the FSAR (Figures 5-19 and 5-21) thereby distributing load to the structure based on composite action between the two materials.

4.1 Applicable codes for the design of the containment building:
FSAR Section 5.2.3.1: The reactor building has been designed under the following Codes:

a. Building Code Requirements for Reinforced Concrete, ACI 318-63.
b. Specifications for Structural Concrete for Buildings, ACI 301-66 except as modified in the design and Quality Control of this building.
c. Specification for the Design and Erection of Structural Steel for Buildings, 1963, AISC.
d. ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels; Section VIII, Unfired Pressure Vessels; Section IX, Welding Qualifications (applicable portions).
e. Specification for Design and Construction of Reinforced Concrete Chimneys, ACI 505-54.

FSAR Section 5.2.2.4.1: The reactor building liner and penetrations conformed in all respects to the applicable Sections of ASA N 6.2-1965. The personnel access locks, the portion of the equipment access door extending beyond the reinforced concrete shell, and the internal primary pressure boundary of all penetrations conformed to the requirements of the ASME Boiler and Pressure Vessel Code Section III Class B.

4.2 The following FSAR Sections are pertinent to the liner plate:
a. Section 5.2: The prestressed concrete shell ensures that the structure has an elastic response to all loads and that the structure strains within such limits so that the integrity of the liner is not prejudiced. The liner has been anchored to the concrete so as to ensure composite action with the concrete shell.
b. Section 5.2.4.1.1: The physical properties of the steel liner and the concrete shell, the geometry of the structure, and the breakdown of the shell into various parts, together with shell type, number of segments, and shell layers per part as used in Kalnin's Program are shown in Figure 5-19.
c. Section 5.2.5.2.1: A further definition of "load capacity" is that deformation of the structure which will not cause (compressive) strain in the steel liner plate to exceed 0.005 inch/inch, nor average tensile strains to exceed that corresponding to the minimum yield stress.
d. Section 5.2.5.2.2: The liner has been designed to support dead load and wind loads during the erection period. Normal operating and accident load requirements are described in Sections 5.2.1 and 5.2.3. The liner has been designed so that the critical buckling stress is greater than the proportional limit of the steel.

4.3 The following DBD 11 Sections are pertinent to the liner plate:
a. DBD 11, Section “Containment Liner” - Acceptance Criteria: Strain not exceeding 0.005 in/in (compression), nor that corresponds to minimum yield (tensile) strain.

4.4 ACI Code Requirements for Containment Liner Plate Analysis:
a. ACI 318-63 Liner Plate Provisions: This is the Code of Record for the design of the CR3 containment; however, it is primarily focused on the design of standard reinforced concrete beams, columns, slabs, etc, not nuclear containment buildings.

b. ACI 349-72: Section 2.6.3 (CR3 is not committed to this code; it is listed here for information only): "The analysis shall consider the interaction of the liner plate and the concrete structure. If a case is found in which the action of the liner results in lower concrete or reinforcement stresses, then the presence of the liner shall be disregarded for that particular case, with the exception of the test condition".

The ACI 349-72 report represents the first published industry standard for concrete containment structures, and was available in draft form for 6 years prior to 1972. The design of many concrete containment structures in the late 1960s and early 1970s was guided by drafts of ACI 349-72 rather than relying solely on ACI 318 which as previously noted was not written specifically for containments. Following ACI 349-72, a joint committee of ASME Section III/Div2 and ACI 359 took over responsibility of concrete containment structures from the ACI 349 Committee.

4.5 ASME Section III, Division 2 (2007) - Requirements for Metallic Liner Plates: (CR3 is not committed to this code for liner plate design; it is listed here for information only)
a. Section CC-3121: The liner shall not be used as a strength element. Interaction of the liner with the containment shall be considered in determining maximum strains.
b. Section CC-3122 (a): The liner shall be designed to withstand the effects of imposed loads and to accommodate deformation of the concrete containment without jeopardizing leak-tight integrity.

5.0 How was the Liner modeled in the ANSYS Finite Element Model?
Refer to Calculation S10-0002, Rev.0, Finite Element Model Description (Reference 1), for a detailed explanation of how the liner was included in the FEM. 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 elastic-plastic shell elements on the inside face of the containment building. Each node has 6 degrees of freedom. 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 (Ref. 1, Section 3.1.4). The liner plate thickness is increased to 1.125 inches around the equipment hatch.

6.0 Justification for Including the Liner Plate in the Finite Element Model:
FSAR Section 5.2 states that "The liner has been anchored to the concrete so as to ensure composite action with the concrete shell". Since the FSAR recognizes composite action between the liner and concrete, including the liner plate in the containment finite element model is reasonable and results in a true representation of the stiffness and corresponding load distribution in the structure from composite action between the two materials.

Table 1 shows how the identified design basis loads (evaluated in the vertical direction only for this example), when applied to the containment structure, distribute as either tension or compression loads to the concrete wall and/or liner plate.

(see request folder for table)

The following conclusions can be made based on Table 1:

1. Initial prestress and dead load are distributed to both the concrete and liner plate based on their relative stiffness to each other, thereby conservatively reducing the compressive force in the concrete, i.e. less compressive force to offset the applied tension forces on the section.
2. Accident pressure applies a tension load to both the liner plate and concrete partially relieving some of the pre stress (compression) in both. Including the liner in the model results in an un-conservatively higher prestress load in the concrete when considering accident pressure since the pressure load is shared between the liner and concrete.

3. Accident temperature results in the liner quickly heating up; however, expansion of the plate is restrained by the concrete due to the embedded steel angles and concrete's lower coefficient of thermal conductivity. The liner plate goes into compression which results in an equal and opposite reaction in the concrete, i.e. a tension force is transferred to the concrete. Including the liner plate in the model conservatively results in adding tensile stress to the concrete.
4. The resulting compressive force in the liner is a summation of the compressive force due to dead, prestress and thermal minus the tensile force resulting from accident pressure. Note that the resulting force in the liner plate is always compressive (Refer to calculated liner and concrete stresses in Attachment 1) for all design basis load combinations.

The effects of Items 1 thru 4 can be equated as follows:
With reference to Table 1 the maximum compressive stress in the liner plate in the vertical direction based on the accident load case of (D + F + Ta + 1.5 Pa) is:

D liner + F liner + Ta liner – 1.5 Pa liner ≤ 36 ksi(Equation 1)

In the above expression the residual compressive stress that can be assigned to accident temperature (Ta liner) is limited to:

36 ksi - (D liner + F liner – 1.5 Pa liner)

An example may clarify the above approach using typical values (from Attachment1) for vertical stress in the CR3 containment walls:

D liner = -0.443 ksi, F liner = -6.369 ksi, Pa liner = 4.465 ksi, Fy liner = 36 ksi

Residual compressive stress available to accommodate accident thermal is:

36 – 0.443 – 6.369 + 1.5 x 4.465 = 35.89 ksi (compression)

The results above indicate that including the liner plate in the model does not substantially reduce the compressive stress available in the liner plate to accommodate accident thermal loads. However, including the liner plate in the model will result in a worst case loading condition for the concrete due to the reduced compressive load in the concrete that results from sharing the prestress and dead load between the liner and concrete, and increased tensile load (in the concrete) due to liner plate thermal expansion. These effects would be partially offset by the reduced pressure load on the concrete since the pressure load is now shared between the liner and concrete.

Table 2 compares the accident vertical load distribution to:
(i) The containment wall including the liner plate in the model.
(ii) The containment wall without considering the liner plate in the model.

(see request folder for table)

How to interpret this Table: In Column 1, including the liner plate in the model causes prestress, dead load and accident loads to be distributed between the concrete and liner. In Column 2, excluding the liner from the model results in the concrete carrying all these forces. Accident thermal is assumed acting on both models, with and without the liner included.

7.0 Liner Plate as a Strength Element
As previously noted in Section 6.0 including the liner plate in the finite element model reflects the real life structural stiffness matrix for the composite structure resulting in a more accurate distribution of applied loads. The inherent stiffness of the liner plate is based on its area (A) and its elastic modulus (E).

Although ASME Section III, Division 2 is not part of the CR3 design basis for the steel lined concrete containment, it does provide a good reference for gaining insight into the current state of the art approach to containment design. Section CC-3121 states:

- "The liner shall not be used as a strength element". In Bay 3-4, when evaluating the flexural, axial and shear strength of any particular cross-section of the containment shell against the moments and forces output from the ANSYS finite element model, the liner plate is included in the model for distribution of forces but not included as reinforcement. The strength of the containment shell is evaluated by both Working Stress Design and Ultimate Strength Design methods per the requirements of the FSAR and ACI 318-63. In all of these evaluations the liner plate is ignored and not considered as additional steel reinforcement.
- "Interaction of the liner with the containment shall be considered in determining maximum strains". The ASME Code recognizes the importance of including the liner plate stiffness in the overall structural containment FEM model and its importance in distributing loads from which corresponding strains can be calculated. Based on this conclusion and other similar requirements in the FSAR the liner plate has been included in the ANSYS finite element model of the delaminated and repaired CR3 containment.

7.1 Liner Plate Behavior in general during Normal Operating Conditions:
FSAR, Section 5.2.3.3.1 requires that membrane tensile stresses in the concrete are eliminated for normal design loads, hence the liner is never required to resist tension and never acts as a strength element in resisting tensile stresses for design loads.

7.2 Liner Plate Behavior in general during Accident Loading Conditions
For the general case, under accident loading conditions the liner plate actually acts as a loading element not a strength element as shown in Table 2 and Attachment 1. Since the concrete wall and liner plate are considered a composite section, the concrete will restrain the thermal expansion of the liner plate resulting in the liner plate exceeding its compressive yield strain and imposing a tensile force in the concrete as discussed in Section 6.0. In this case the liner physically cannot act as a strength element since it is in compression and has exceeded its compressive yield strain thus ensuring that the liner plate would not act as a strength element. A detailed ANSYS FEM analysis is being performed that will report liner plate stresses and strains, and will identify any anomalies in the liner plate strains which may occur in and around the SGR opening in Bay 3-4. Note that per FSAR Section 5.2.5.2.1 the liner plate average tensile strains must not exceed that corresponding to the minimum yield stress. Calculation S10-0058, Rev.0, Evaluation of Vertical Cracks outside the SGR Bay (Reference 2) shows that during design basis accident pressure the liner will always stay in compression. Since the post-accident liner never goes into tension even when the benefit of thermal expansion is ignored, it can never provide any strength in resisting applied tensile loads.

8.0 Effects on Liner Plate due to Detensioning and Retensioning Tendons:
Since the liner plate has been included in the ANSYS FEM of the containment shell, the effects of detensioning and retensioning have been included in the finite element analysis. Including the liner plate per design results in an accurate distribution of the prestress forces. Refer to Calculation S10-0004, Tendon Detensioning Calculation, for effects on liner due to detensioning, and Calculation S10-0012, Stresses around the SGR Opening Due to Design Basis Load Cases, for effects on liner during repair and retensioning.

9.0 Original Gilbert Associates Design Basis Calculations:
A search was made of the original containment calculations prepared by Gilbert Associates to identify if the liner was included in their original Kalnins model of the containment shell. Several instances were found that indicated that the liner had been included in the Kalnins analytical model. Refer to Attachments 2, 3, 4 and 5 for more information. Also, both models in the FSAR (Static and dynamic) include the liner plate (Figures 5-19 and Figure 5-21).

References:
1. Calculation S10-0002, Rev.0, Finite Element Model Description
2. Calculation S10-0058, Rev.0, Evaluation of Vertical Cracks outside the SGR Bay
3. Gilbert Calculations 1.01.19 and 1.01.22

Attachments: (see request folder for attachments)
1. Sample Analysis - Accident Load Distribution to Concrete and Liner Plate
2. Pages from Gilbert Associates, Inc., Calculation 1:01.19
3. Pages from Gilbert Associates, Inc., Calculation 1:01.21
4. Pages from Gilbert Associates, Inc., Calculation 1:01.22

Supplemental Response:

The initial response to Question #67 included a hand calculations performed in Mathcad to prove that, for the general case, including the liner plate in the containment shell ANSYS finite element model (FEM) resulted in lower compressive stresses in the concrete, and that the liner plate remains in compression for the critical load case of Dead Load + Prestress +1.5 Design Pressure, and was therefore conservative. As a further check, an ANSYS FEM was used to evaluate the effects of removing the liner plate from the ANSYS model on concrete stresses for all design basis loads.

To address this request, MPR Calculation 0102-0135-23, Containment Finite Element Analysis Evaluation with Liner Removed (Reference 1), was prepared. This calculation determined the impact of explicitly modeling the liner stiffness on concrete tensile and compressive stresses in Bay 34. The evaluation was performed on the same 180-degree symmetric three-dimensional FEM as described in MPR Calculation 0102-0135-04, Finite Element Model Description (Reference 2) and MPR calculation 0102-0135-09, Stresses from Design Basis Load Cases Around the SGR Opening (Reference 3). However, the FEM used for the Reference 1 calculation removed the liner plate from the model. Removal of the liner plate from the model necessitated some additional changes to account for the liner thermal accident loading and to transmit the crane moment loads to the containment wall. These changes are discussed in more detail in Reference 1.

Reference 3 evaluates the containment under all design basis load combinations with the liner plate included in the ANSYS FEM. The results show that the 0.95D + Fa.eol + 1.5P +Ta load combination results in the maximum concrete tensile stresses in the area of the SGR opening and the 1.05D + Fo.rts +/- Eh +/- Ev load combination produces the maximum compressive stresses. Reference 1 determined the concrete stresses for these same two (actually three load combinations due to the sign reversal in the seismic load combination) load combinations with the liner plate removed from the FEM as previously discussed.

Reference 1 includes a tabulation of the results for the three load combinations for End-Of-Life “With Liner”, and End-Of-Life “No Liner”. The difference in the two sets of stresses is then determined by subtracting the End-Of-Life “With Liner” from the End-Of-Life “No Liner” with the results tabulated in Tables 7-19 thru 7-25 and 7-44 thru 7-49. The results show that removing the liner’s load carrying capacity from the FEM causes little change in the stresses in the repair region.

To better understand the overall impact on the forces and moments in Bay 34, and the corresponding impact, if any, to the steel reinforcing design documented in Calculation S10-0030, Rev. 1, Reinforcement Design for Delaminated Containment Wall (Reference 4), AREVA Calculation 32-9147814 is being developed (Reference 5). Reference 5, Attachment M contains three sets of data:

(i) Global model results without the liner
(ii) Global model results with the liner
(iii) (Global model results without the liner) – (Global model results with the liner)

The difference in stresses reported in Item (iii) is added to the Refined Global stresses reported in Appendix N of Calculation S10-0030 (Reference 4) and the summation is reported in Reference 5, Appendix N. . Reference 5 then envelopes the Global Model results without liner described in Item (i) above with the results contained in Reference 5, Appendix N and tabulates the results in Appendix A. The purpose of enveloping these two sets of results (data) is as follows:

- Load Combination 1 (0.95D + Fa.eol + 1.5P) puts the containment under tension; therefore the maximum value from the two sets of data is determined.
- Load Combination 2 (1.05D + Fo.rts +/- Eh +/- Ev) puts the containment shell under compression; therefore the minimum value from the two sets of data is determined.

The results in Appendix A are then averaged as described in Section 6.3 of Reference 5 and the averaged vertical and hoop direction stresses are reported in Appendix B and C. Resultant forces, axial force and bending moment design, shear force design, and Interaction diagrams in the vertical direction are provided in Appendix D, Appendix F, Appendix H, and Appendix J respectively. Resultant forces, axial force and bending moment design, shear force design, and Interaction diagrams in the hoop direction are provided in Appendix E, Appendix G, Appendix I, and Appendix K respectively.

No reduction in conservatism was taken in this comparison. For example, the thermal gradient effects were based on accident temperature utilizing the same approach as was used in Reference 4, i.e. using the methodology recommended in ACI 505-54, Specification for the Design and Construction of Reinforced Concrete Chimneys. The resulting thermal moments (Reference 4, Section 6.1) were then added to the primary moments reported in Reference 5, Appendix A.

Reference 5 concluded that removing the liner plate from the model decreased or increased insignificantly in general the tension stresses in the wall for Load Combination 1. Load Combination 2 resulted in axial compression and compressive bending stresses that in general increased in magnitude. These comparison results are reasonable since the liner plate takes a significant share of the prestress force when included in the model. Reference 5, Section 6.2 contains a detailed account of the comparison between the results from the containment model including the liner plate and those from the containment model excluding the liner plate. Reference 5 evaluated the design of the containment wall in Bay 3-4 for axial tension and bending in all critical areas, for shear, and for combined axial compression and bending to assure that the section capacity is not exceeded in any direction.

Conclusion:
Removing the liner plate from the model will generally decrease or increase insignificantly the tension stresses in the wall for Load Combination 1, and increase in general the axial compression and compressive bending stresses for Load Combination 2. The reinforcement design provided by Reference 4 has been evaluated for axial tension and bending, shear, and combined axial compression and bending in all critical areas and found to be acceptable.

Based on the 1.5P load case, the maximum concrete strain from the global model without the liner is within 5 μ in/in (4%) of the maximum concrete strain from the global model with the liner included, therefore, the liner strains are effectively the same as those reported in the S10-0012 calculation (Reference 3).

References:

1. MPR Calculation 0102-0135-23, Rev.0, Containment Finite Element Analysis Evaluation with Liner Removed (not approved – to be included in S10-0054).
2. Calculation S10-0002, Rev.0 (MPR Calculation 0102-0135-04), Finite Element Model Description.
3. Calculation S10-0012, Rev.1 (MPR Calculation 0102-0135-09), Stresses from Design Basis Load Cases around the SGR Opening.
4. Calculation S10-0030, Rev. 1, Reinforcement Design for Delaminated Containment Wall.
5. AREVA Calculation 32-9147814, Evaluation of the CR3 Containment Repair Design without the Liner (not approved – to be included in S10-0054).
6. Calculation S10-0054, Containment Analysis with Liner Removed (not issued - this calculation will include References 1 and Reference 5).

Misc Notes:

Response By:	Paul Fagan		
Reviewed By:		**Date Response Provided:**	11/16/2010
Status:	Closed	**Date Closed:**	12/17/2010

Request Number: 68

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Question

Request: Refer to Slide #75 of the 11/20 public meeting presentation. Slide states: "Run comparison to original design building elastic design results."

Explain how you plan to evaluate your analysis results for design basis loads and load combinations against acceptance criteria in accordance with the code of record, i.e., ACI 318-63, in the FSAR. How would you process your analysis results to perform code checks for stresses, strains, displacements or other applicable design basis acceptance criteria for concrete, rebar, liner and prestressing tendons? How is reinforcement being accounted for in your design basis evaluation?

The slide only indicates evaluation for controlling factored load combinations. Are there not service or other load combinations in the design basis with a different set of acceptance criteria that needs to be documented? How would your calculation document the design basis of the modified containment following repair of the delaminated condition?

How will stresses in the concrete and rebar be determined from the ANSYS analysis? Provide your approach to performing the finite element analysis and design checks in support of the Design Basis Analysis considering the various interim configurations associated with the creation and restoration SGR construction opening, the delaminated condition and the associated repair?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The analyses of the delamination and associated repair activities on the containment structure can be divided into two areas; Bay 34 and Bays other than Bay 34. These analyses were performed for service loads and design basis loads identified in the FSAR and Design Basis Document (DBD). The results were evaluated based on acceptance criteria consistent with the Crystal River 3 (CR3) codes of record. A series of calculations was performed to assess the stresses in the concrete and reinforcing steel for the various configurations of the repair effort. These calculations and details regarding their scope, intent, and methodology are listed below for the two areas in question:

Bay 34 (SGR Repair Area):

- Calculation S10-0002, Finite Element Model Description, documents the ANSYS finite element model of the CR3 Containment Building. The model was developed to analyze containment restoration and design basis loading conditions.
- Calculation S10-0032, Limiting Load Cases, determines the limiting load cases for the containment building based on the normal operating and accident load combinations identified in the FSAR and the DBD. The limiting load cases apply to an evaluation for limiting tensile stress in the concrete of the containment building.
- Calculation S10-0012, Stresses around the SGR Opening due to Design Basis Load Cases, reports membrane, membrane plus bending, and shear stresses in the vicinity of the repaired steam generator replacement (SGR) opening in the detensioned state under deadweight and remaining prestress, and in the fully repaired and retensioned state for limiting design basis load cases contained in Calculation S10-0032, in addition two load cases that produce maximum compressive stress, and the unfactored accident pressure load combination and test pressure load combination.

• Calculation S10-0015, Refined Containment Finite Element Model Evaluation, documents a refined ANSYS finite element model of the Crystal River Unit 3 (CR3) containment and compares the stress results from this analysis to the results from the finite element model used in calculation S10-0012. The model in calculation S10-0012 uses simplified vertical and hoop tendon profiles in the region of the equipment hatch as well as a simplified equipment hatch opening geometry. The refined model was developed to evaluate the effect of modeling actual tendon profiles and containment geometry in this region on the stresses in the 42-inch thick concrete wall between buttresses 3 and 4.
• Calculation S10-0021, Concrete Radial Reinforcement. The purpose of this calculation is to qualify the use of #5 reinforcing bars with a 135° hook as an anchorage system that will connect the original concrete to the new replacement concrete and the outer reinforcement mat. The anchor system will be qualified for the calculated radial tension developed between the two faces of concrete at the vertical plane of the hoop tendons as developed in Calculation S09-0054.
• Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall, provides the concrete and reinforcement design per ACI 318-63 and the FSAR for Bay 34 based on the results contained in calculation S10-0012 and S10-0015 and the ANSYS Finite Element Model (FEM) described in calculation S10-0002. Five load factor (accident) load combinations are evaluated per the ultimate strength design method (USD) and seven service load combinations are evaluated per the working stress design (WSD) method. The effect of operating or accident thermal gradient were not included in the analysis performed in calculations S10-0012 and S10-0015. These effects have been analyzed separately in calculation S10-0030 and the resulting thermal moments added to the primary loads results. The reinforcement arrangement is shown on drawings 421-354 thru 421-361.

Bays Outside the SGR Repair Area:
The extent of influence on stresses outside Bay 34 due to the delamination and consequent repair will be evaluated in Calculation S10-0057, Construction Impact on Containment Stress under Design Basis Loads. This calculation compares the containment stresses outside of Bay 34 due to design basis loads, including deadweight and prestress, before the creation of the SGR access opening to those stresses that exist after the completion of the containment repair, and at end of life. The evaluation of these stresses is ongoing. Any increase in stress will be evaluated per the design basis acceptance criteria and will be based on the existing concrete having zero tension capacity.

References:

1. Progress Energy Calculation S10-0002, Rev.0, Finite Element Model Description.
2. Progress Energy Calculation S10-0012, Rev. 2, Stresses around the SGR Opening due to Design Basis Load Cases.
3. Progress Energy Calculation S10-0030, Rev.0, Reinforcement Design for Delaminated Containment Wall.
4. Progress Energy Calculation S10-0032, Rev. 0, Limiting Load Cases.
5. Progress Energy Calculation S10-0057, Draft, Construction Impact on Containment Stress under Design Basis Loads (MPR Calculation 0102-0135-26, Rev. 0).

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	9/17/2010
Status:	Closed	**Date Closed:**	10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 69

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Question

Request: Refer to Slide #74 - "Planned Analysis Steps" of the 11/20 public meeting presentation. Footnote (1) against "Delamination states "Analysis will consider time of delamination and specific concrete properties."

Since the final root cause analysis results will not be known until later, do you plan on running two different cases with regard to timing of delamination at this time? Specifically, with regard to making a decision on the number of tendons that will be required to be detensioned prior to repair and retensioned following repair.

Regarding the bullet that states: "SAVE path dependent model for starting point to Run 5 controlling design cases." As you go through the planned analysis steps, explain how your analysis model or ANSYS software is capable of starting the next analysis step using the deformed configuration of the previous step as the initial conditions for the next analysis step?

Are you planning to use the same concrete mix design as for the SGR construction opening in implementing repair of the delaminated area? How are properties of the new concrete being incorporated into your analysis?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Sequence:
Analysis that supports additional detensioning in 2010 was modified to account for best information on delamination versus detensioning performed in 2009. Calculation S10-0004, Tendon Detensioning Calculation, includes the effect of delamination after tendons are detensioned and concrete has been removed. Calculation S10-0012, Stresses around the SGR Opening due to Design Basis Load Cases, includes the same sequence but also addresses a study where delamination occurs before the concrete in the SGR opening is removed. The effect on the detensioned state between delamination occurring after concrete removal or prior to concrete removal was determined to be negligible.

Restarts:
To simulate the load history described in these calculations S10-0004 and S10-0012, deformations from the previous case are SAVEd and then RESTARTed for the next load sequence. This ensures that the load history of the structure includes stresses created during each of the construction evolutions.

Concrete:
The same concrete mix design planned for the SGR opening is being used for the larger repair area. For computer analysis, only Young's Modulus and Poisson's Ratio are included. The aging effects (creep and shrinkage) of concrete on tendon forces were determined. Possible variations in Young's Modulus are being addressed in a sensitivity analysis. Concrete strength in the repair area is evaluated coincident with design of additional reinforcing steel.

References:
S10-0004, Tendon Detensioning Calculation

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

S10-0012, Stresses around the SGR Opening due to Design Basis Load Cases

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	9/23/2010
Status:	Closed	**Date Closed:**	10/19/2010

Request Number: 70

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Question

Request: With reference to 11/20 public meeting presentation, Slide 65 - shows approximation in Equipment Hatch modeling; and Slide 34 - shows that the delaminated conditions extends to above the EQ hatch area; slide 35 shows hoop tendons that wrap around EQ hatch. Further, there are also removed vertical tendons that wrap around EQ hatch. If your detensioning/retensioning scheme involves tendon elements that influence forces in the EQ hatch area, how do you plan to address it in your design basis model? Describe any plans to refine your model around the EQ hatch area.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The detensioning/tensioning scope does involve vertical tendons that wrap around the equipment hatch. The detensioning/tensioning schedule has one horizontal tendon (53H17) that is impacted by the equipment hatch. A decision has been made to specifically model in the tendons that wrap around the equipment hatch in a submodel. This submodel has been run for the controlling load case (1.5 P + Ta) and the results around the steam generator opening have been compared with the results of the design basis model. The comparison of the two models yields compatible results. The comparison is being documented in a calculation, Equipment Hatch Submodel. (MPR calculation 0102-0135-12 and Progress Energy Calculation S10-00015)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 71

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to slide 58 of the 11/20 public meeting presentation - describes a 180 degree symmetric model.

Please confirm whether, for your analysis, the explicitly developed 180 degree model is extruded to 360 degrees for your runs or not.

Please confirm if there are any unsymmetric containment features that may not be adequately represented in a symmetric model but may affect the response of the affected area.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

It is not anticipated that the 180 degree model will be extruded to a 360 degree model for any of the stress analysis.
Unsymmetrical containment features will be individually evaluated to determine if the asymmetry might significantly affect the stress analysis. These evaluations may or may not involve local analysis or modeling to account for the asymmetry.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 5/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 72

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Slide 74 (and 76) of the 11/20 public meeting presentation. The first three planned analysis steps are: (i) Dead Load + Tendons; (ii) Remove Hoop + Vertical Tendons in SGR opening; and (iii) Remove SGR opening. Provide stress and deformation plots for the area in and around the vicinity of the SGR opening (between Buttresses 3 & 4 from above the EQ hatch to below the ring girder) for each of the above configurations for the Dead + Prestress load combination.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Relevant stress and deformation plots are included in Calculation S10-0012, “Stresses Around the SGR Opening due to Design Basis Load Cases.” Although included in the analysis, the three particular load steps requested are not reported graphically.

REFERENCES:
Calculation S10-0012, “Stresses Around the SGR Opening due to Design Basis Load Cases.” (see request folder for reference)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/5/2010

Status: Closed

Date Closed: 10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number:	73		
Individual Contacted:	Garry Miller	**Date Contacted:**	12/2/2009
Requestor/Inspector:	George Thomas	**Category:**	Information Request

Request:

Refer to Slide 81 of the 11/20 public meeting presentation with regard to Post Repair Testing.

Provide the name and credentials /qualifications of the designated Responsible Engineer, in accordance with ASME Section XI, Subsection IWL, for repair/replacement of the CR3 containment structure related to the SGR project and the Containment Delamination project. Provide the date the individual was designated as the Responsible Engineer.

Second bullet on the slide states: "Concrete exterior will be visually examined prior to pressurization and following depressurization." Third bullet states: "Evaluating other additional instrumentation based on the final repair that is implemented, and as driven by: root cause analysis." For the major containment repair/replacement activity involved at CR3, describe how the post-repair system pressure testing would meet the requirements of IWL-5000, and specifically provide verification of the containment structural integrity under accident pressure and corresponding structural behavior as predicted by the design basis analysis.

The response is incomplete/inadequate as indicated below. Provide a complete response to address these concerns. Also, confirm whether or not the design basis accident pressure and/or the containment design pressure was affected by the extended power uprate being implemented during/following the RF16 Outage.

1. The information provided with regard to qualifications and credentials of the designated IWL Responsible Engineer for the SGR Project, does not indicate nor provide evidence of basic qualifications required by IWL-2320 for an individual to be designated the Responsible Engineer. Provide evidence (such as PE registration) and information of required qualifications stated in the first paragraph of IWL-2320 (ASME Section IX, 2001 Edition with 2003 Addenda) for the designated Responsible Engineer. Also, include a resume with educational qualifications and experience of the individual.

2. There was no response provided with regard to the designated Responsible Engineer for the Containment Delamination Project, the person's qualifications and credentials and date of designation. Provide the requested information for the designated Responsible Engineer for the Containment Delamination Project.

3. For the major repair of the extensively delaminated condition of the CR-3 containment involving new design/construction features, the response provided with regard to examinations during the containment pressure test does not meet the requirements and intent of IWL-5250. Just performing visual examination of the repaired concrete surfaces prior/during/after the test, without performing structural response measurements and additional examinations, will not demonstrate the quality and adequacy of the repair (i.e. the repaired containment has not delaminated again) nor will it provide a verification of structural response/behavior as expected and predicted by the design basis analysis. Further, there will not be data available to compare to a previous benchmark test (such as original SIT) to fully demonstrate structural integrity of the repaired containment.

References:

Response Assigned to: Paul Fagan **Date Due to Inspector:**

Response:

Updated Response:

1. Provide the names and qualifications of the designated Responsible Engineers, in accordance with ASME Section XI, Subsection IWL, for the SGR project as well as the Containment Delamination project, as well as the dates of designation as the Responsible Engineers.

The designated IWL responsible engineer for the SGR project was John Holliday. John Holliday and Joe Lese are both filling the IWL responsible engineer position for the Containment Delamination project.

The effective date of John Holliday's designation is the date of his training guide completion (7/14/09). Note: NCR 373018 documents that the incorrect training guide was initially used for documenting the IWL responsible engineer qualification (Containment Inspection Program Training Guide vs. Containment Inspection Responsible Engineer Training Guide). The primary difference between the two training guides is the Registered Professional Engineer requirement. The correct training guide has since been completed.

Joe Lese has been the CR3 designated IWL responsible engineer since the program was first implemented, although the training guide was not completed until 9/10/09 (the training guide did not exist at the time of original designation).

Completed training guides, resumes, and PE registrations for John Holliday and Joe Lese are attached. (Note – Joe Lese's resume has not been updated since he started at Progress Energy (Florida Power at the time. He has been a Structural Engineer in the Design Engineering organization from 1989 to present).

(see request folder for resumes and qualifications)

2. Describe how the post-repair system pressure testing would meet the requirements of IWL-5000, and specifically provide verification of the containment structural integrity under accident pressure and corresponding structural behavior as predicted by the design basis analysis.

IWL-5000 requires a pressure test at Pa, detailed visual examination of new concrete during the test, and other examinations / measurements as specified by the Responsible Engineer (RE). These examinations will be performed in conjunction with the integrated leak rate test (ILRT) at accident pressure. We will also perform a structural integrity test (SIT) at 1.15 x the containment design pressure in accordance with ASME Section III, Division 2, Article CC-6000. Crack mapping, radial and vertical displacement measuring, and strain monitoring will be performed per CC-6000 and as specified by the RE and as described in the Response to NRC Question 61.

3. Confirm whether or not the design basis accident pressure and/or the containment design pressure was affected by the extended power uprate being implemented during/following the RF16 Outage.

The containment design pressure is unaffected by the power uprate. The design accident pressure will be no higher than the current accident pressure.

Misc Notes:

Response By: Craig Miller

Reviewed By: **Date Response Provided:** 8/18/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Status:	Closed	**Date Closed:**	8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

Request Number: 74

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: Anthony Masters

Category: Question

Request: Refer to photos on Slide 14 of the 11/20 public meeting presentation.

Explain the gap between the liner and the concrete? Have you verified how far it goes?

It is our understanding that there is bulging in the containment liner with air voiding between liner and concrete at several locations all around between approximate EL 180 and 225 ft; and that it was dispositioned as construction/fabrication errors that existed prior to concrete pour. If this existed prior to original concrete pour, explain how there is voiding between the liner and concrete. What was the acceptance criteria used to evaluate this? Provide the engineering evaluation for accepting the bulging as-is and explain how this evaluation is consistent with CR3 current design basis.

Supplemental Request:

Do calculations by Structural Integrity and Associates (Calculation S10-0046) capture the load history of the structure? What calculation will be used to document the final condition of the liner plate and acceptance with regard to design basis?

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

The referenced photo on slide 14 of the 11/20 public meeting presentation shows the containment wall facing buttress 3 in the excavation area where concrete was removed to the liner plate. The entire expanded SGR opening was inspected by CR3 Containment Repair Project engineers and as-found measurements of gaps between the liner plate and concrete were provided by SGT engineers. The inspections resulted in the incorporation of repair instructions into EC 75219. The repair instructions provided for the removal of concrete in the areas specified by PGN engineering based on the gap type. There are two conditions observed during the inspections, 1) small gap with limited depth caused by washout during hydrodemolition and 2) larger gaps with depths as deep as the next embedded stiffener angle. The larger gaps (i.e., bulges) are being addressed by CR3 plant engineering and will not be addressed in this Containment Repair Project request response.

SGT Work Package 3732C (implementing EC 75219) contains the expanded SGR opening pre-excavation as-built data of the concrete to liner plate interface. This information and examination of the interface was used to identify areas that required concrete removal. The areas that experienced hydrodemolition washout did not require concrete removal due the gap width and depth. The area will be filled with fresh concrete during placement. Concrete was removed in areas with gaps not caused by washout to a depth specified in EC 75219. The excavation removed only enough concrete to ensure flow of fresh concrete behind the existing concrete towards the next liner stiffener angle. There are no acceptance criteria based on the fact that the gaps will be filled eliminating the gaps.

Additional Response (10/18/10):

Calculation S10-0046 has been developed to evaluate bulges in the CR-3 liner plate. This calculation was directed at determining an apparent cause for the bulges and establishing an analytically based acceptance criterion for the bulge within the CR3 design basis. The analyses include FEM modeling of the liner and the associated anchorage to the concrete containment structure.

The apparent cause for the bulges is a combination of elements. The first being original geometrical imperfections in the liner plate at original construction. The calculation demonstrated the liner plate will not deform, as to result in a bulge, under anticipated operational conditions. If the liner is installed with the design radius, backed by concrete, anticipated loadings result in compressive membrane strains with the plate maintaining the installed configuration. If the liner is installed with an imperfect shape, even a small offset into the design radius, design loadings will induce bending, additional deformation and bulging. The design loadings that will result in the additional deformation or bulges into containment include; thermal loading, pre-stress strains and creep strains. The strains account for the gap behind the liner in the area of the bulges, as well as some growth in the bulges over time.

The S10-0046 calculation evaluated the bulges against the original design basis with the original liner plate analysis addressing membrane strain. Additionally bending strains and embedded anchors were evaluated applying the 1975 ASME Code, Section III, Division 2. The original plant construction calculation did not evaluate potential bending strains. The 1975 code year was selected based on its application in CR3's assessment of the dome delamination in 1976. Based on these criterion it was determined a bulge as large as 1.82" would result in strains and anchor loadings that would be acceptable against the current CR3 design basis. The limiting bulge size of 1.82" bounds all of the current liner bulges with sufficient margin to address anticipated bulge growth.

With the ongoing containment repair effort the calculation also took the extra step of evaluating the significance of placing containment opening repair concrete against existing bulges. The calculation made the determination that backing (or partially backing) existing bulges with replaced concrete would not impact the validity of the analysis or the acceptable bulge size.

Supplemental Response (12/8/10):

Bulges are evaluated in Calculation S10-0046, "Liner Bulge Evaluation" (by Structural Integrity and Associates, SIA). Worst case configurations are considered and a threshold for bulge size is established considering the effects that occur due to normal operation and accident conditions. Creep is addressed and a general position is established that the analysis isn't sensitive to changes in the structure. The primary variables in the bulge evaluation are bulge size and thermal loading.

MPR has also evaluated the containment structure and liner for changes that have occurred in the process of Steam Generator Replacement (SGR), further detensioning and concrete repair. The entire load history is captured for analysis of each subsequent step. These analyses address impacts to liner stress / strain but make no specific analysis of the effect of bulges. These analyses have a large margin with regard to liner strain. As discussed in Calculation S10-0054, "Containment Analysis with Liner Removed", inclusion of the liner in the finite element model is conservative because it takes away prestress from the concrete. When worst case temperature and pressure are combined, the prestress is lost to plastic stain as the liner heats. Also, as discussed in the supplemental response to S.I.T. Question 67, the effect of removing the liner from the model is only a 4% change to concrete strain for the 1.5P+Ta load combination. Therefore, liner strain is essentially unchanged. Modeling bulges in the finite element analysis can be approximated by ignoring the liner all together and therefore, the analysis in Calculation S10-0054 bounds the bulged liner condition for effects to the load capacity of the structure. In summary, bulges have an insignificant effect on the response of the structure. This analysis includes the load history.

Current bulges are bounded by the acceptance criteria in the SIA analysis. NCR 377992 provides an investigation that evaluates the as-found condition. The NCR does not describe actions that are needed to ensure that conditions are acceptable in the future. Those actions are driven by the IWE program. EC 75221 includes steps to validate the effect of retensioning on bulge size by measurement and evaluation of a representative sample before initiating Structural Integrity Test (SIT) pressurization. EC 75221 also includes requirements to perform a complete baseline scan after completion of the SIT.

A summary evaluation that addresses the points in this supplemental response is being added to EC 75221. It will address interactions of Calculations S10-0046 and S10-0054 and summarize monitoring activity for retensioning, SIT and future surveillances. The IWE program is being modified by EC 75221 to ensure continued acceptability of bulges in future inspections.

Misc Notes: Added photos on liner bulges and paint spall areas above the SGR opening (post hydro, prior to liner cut) to NRC Folder for Question #74 per NRC verbal request.

Response By:	Paul Fagan		
Reviewed By:		**Date Response Provided:**	9/2/2010
Status:	Closed	**Date Closed:**	3/1/2011

20-Apr-11 7:56:36 AM

Request Number: 75

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Describe your plans [PII] for finite element simulation of the delamination to confirm the root cause(s)?

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Multiple finite element analyses are being performed to confirm the root cause. These include the use of the computer code Merlin to perform a 2-D simulation of a vertical cross section of the wall, and Abaqus to perform 3-D simulations. The models include the various parameters considered in the root cause analysis, including concrete strength, creep, thermal gradients, and fracture energy. This subject was also discussed between PII and the NRC SIT on 1/7/10

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 1/8/2010

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number:	76		
Individual Contacted:	Garry Miller	**Date Contacted:**	12/2/2009
Requestor/Inspector:	Robert Carrion	**Category:**	Question

Request:

Refer to the Refuting evidence for failure mode 2.8 "Inadequate Support of Tendons during Pouring." There are photographs of the SGR opening area that show that the as-found hoop tendon sheathing are all not centered on a vertical line.

What was the design location of the tendon sheathing?

Was the installation of the tendon sheathing out-of-tolerance in the as-found condition (Tendon installation specification must have had a tolerance for tendon sheathing installation)?

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

Tendon design drawings (ie Prescon Corporation Drawing P10A) show the hoop tendon centerline as 67' 8 5/8". It refers to Installation Specification for tolerances.
SP-5844 Oct 21, 1970 - Specification – Installation of Prestressing System Tendon Conduit and Embedded Anchorages (FPC 321-B4.1B) includes conduit tolerance requirements. Paragraph 3:05.2 states "Placement tolerances for the conduit shall be in accordance with section 1504 of ACI 301. The tolerances shall be applied to the dimensions shown on the relevant approved Prescon Corporation and/or Florida Power Corporation drawings and other drawings and lists as developed by the OWNER and/or ENGINEER."
ACI 301-66 section 1504 for Placement and Protection of Tendons and Accessories specifies a tendon placement tolerance of +/- ½" for concrete with dimensions over 2'. This tolerance is applicable for this wall since its design is 42" thick.
FM 2.8 found evidence of inspections that identified tendon placements being out of tolerance and the resolution was to correct the out of tolerance condition. Reference NCR 0043 (FM 2.8 Exhibit 2)
FM 1.8 reviewed added stress from tendons not in same plane. It concluded that while there was evidence of a few tendons being out of plane, they were not out of plane enough to be a contributor to the delamination event.

Misc Notes:

Response By:	Charles Williams		
Reviewed By:		**Date Response Provided:**	
Status:	Closed	**Date Closed:**	7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 77

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: George Thomas

Category: Information Request

Request: Confirm whether "the lack of bond between the smooth tendon sheathing and the concrete" is included as a possible failure mode in the root cause investigation.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The lack of bond between the smooth tendon sheathing and the concrete is not identified as a specific failure mode, but it is included as a condition in failure mode 6.4, “Added Stress from Differences Between Rigid and Flexible Sleeves”. The finite element analyses also recognize this condition, and take no credit for any tensile load carrying strength at the sleeve-to-concrete interface.

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 1/8/2010

Status: Closed

Date Closed: 6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number:	78		
Individual Contacted:	Garry Miller	**Date Contacted:**	12/2/2009
Requestor/Inspector:	Anthony Masters	**Category:**	Question

Request: Considering the delamination and subsequent repair of the CR3 dome during original construction, what non-destructive examination, core boring and/or other appropriate testing was extended to the dome during the current investigation of the containment wall delamination issue to confirm that the 1976 dome repairs remained good? Provide results of the examinations performed on the dome. Also, explain how the results for these examinations would help address/resolve the concerns raised in the previous Requests #1 and #40 with regard to the low spot or depressed area on the dome.

References:

Response Assigned to:	Paul Fagan	**Date Due to Inspector:**	

Response:

The Condition Assessment documented in EC 74801 included Impulse Response (IR) testing of the portion of the dome between buttress 3 and 4 extending to the center of the dome. Core bores were made in support of evaluating the IR data. The results of this initial examination are included in EC 74801.

Additional examinations were performed in EC 75218 as part of the post tendon detensioning testing. The examination included the initial area examined under EC 74801 and was expanded to the portion of the dome between buttress 6 and 1 extending to the center of the dome. This area was examined prior to detensioning and then examined again along with the buttress 3 to 4 dome area, after detensioning, to look for indications of a delamination. The results of this examination are included in EC 75218.

Anomalies were identified during the pre-detensioning examination performed in EC 75218. Based on these results, the dome surface area repaired in 1976 was subjected to an additional IR examination. This examination included IR using a 1 ft x 1 ft test grid in both orthogonal (N-S and E-W) directions. A total of approximately 10,000 points were tested. A total of 30 core samples were removed including those already removed during the previous IR examinations. Visual inspections were performed of each core sample and video scope inspections of each core hole after the core sample was removed. The results of this examination are in CTLGroup Project No. 059176 – Dome Report (to be incorporated in a future revision to EC 74801).

The information contained in the CTLGroup Dome Report was utilized in an evaluation performed by Worley Parsons titled Containment Dome Evaluation, Report No. CR-3-LI-537934-52-SE-0059. This evaluation was initiated based on inspections of the interior surfaces of core holes revealing cracking in the plane of the dome (laminar cracking). The purpose of the engineering evaluation is to determine if this laminar cracking in any way degraded the capability of the structure to perform its design basis functions. Worley Parsons performed a visual inspection and concrete sounding of the dome surface. The overall conclusion reached by Worley Parsons is that the repairs made to the dome structure are intact and performing in accordance with the licensed design basis for the plant and that the dome structure is still in the condition as addressed by the original 1976 Dome Delamination Report. In addition, based on the core hole and crack evaluations, Worley Parsons concluded that there are no significant anomalies, discrepancies, or structural issued which would affect the overall structural integrity of the dome structure. The dome exterior surface was observed to be in good condition with some dome exterior surface irregularities and conditions that represent normal aging of the concrete structure, such as some narrow width shrinkage cracks. This engineering evaluation will be incorporated into calculation S10-0050, Containment Repair Project - Post NDE Dome Evaluation.

Previous Requests #1 and #40 have already been addressed. The conclusions reached by Worley Parsons substantiate the responses provided for Request #1 and #40.

Misc Notes:

Response By: Paul Fagan

Reviewed By:

Date Response Provided: 9/29/2010

Status: Closed

Date Closed: 11/17/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 79

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: Dan Naus / George Thomas

Category: Information Request

Request: Explain how your condition assessment performed in accordance with Procedure PT-407T (NDE testing, core bore sampling, boroscopic examination etc.) provides a reasonable assurance of a comprehensive and accurate determination of the extent of delaminated condition of the containment.

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #79

Misc Notes:

Response By: Martin Souther

Reviewed By:

Date Response Provided: 6/24/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 80

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: Dan Naus

Category: Question

Request: Provide information of the total number of core samples that were sent for petrographic examination for the containment delamination issue. Indicate the labs to which each sample was sent. How did you determine/ensure consistency of the examination and results between the labs? How did you establish that a reasonable number of samples were sent for petrographic examination?

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

There have been a total of seven core samples that received some form of petrographic examination. The core identification numbers and test labs are:
1.Core 5MACTEC (1/2 of Core 5 tested at MACTEC)
2.Core 5CTL (1/2 of Core 5 tested at CTL)
3.Core 6Photometrics
4.Core 7MACTEC
5.Core 18Photometrics
6.Core 19Photometrics
7.Core 87MACTEC
MACTEC and CTL performed petrographic examinations in accordance with ASTM C 856. Photometrics evaluated similar conditions and attributes as those evaluated under the ASTM standard, but used tools and techniques more frequently used in material science, e.g., scanning electron microscope (SEM) and micro-hardness examinations that are more thorough. Progress Energy did not provide any directions that would influence how a particular test or examination was performed, other than convey the main objective of the particular examination (i.e., determine age of the break). The purpose of using multiple labs was to obtain independent results; therefore there was no explicit effort to ensure consistency in the examination techniques or results.
Note that not all samples were examined for fracture age determination. For example, Core 87 was taken from the containment dome (area repaired in 1976). The purpose of the petrographic examination on this sample was to compare the aggregate from the dome to the aggregate from the wall.
The number of samples that received petrographic examinations is believed to be adequate based on the consistent results obtained from the various labs and the diversity of the sample locations.

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 12/7/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 81

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: Dan Naus

Category: Question

Request: According to MacTec petrographic report dated November 11, 2009, limited observations were to be performed on sample 21270A (Core #2) which was used as a control sample. However, there is no discussion of how it was used. Also, it does not appear that any results from these observations were reported. What examinations were performed on this sample, what were the results and where is it documented?

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Per discussion with the MACTEC petrographer, the lab did do some limited observations on Core 2, but I did not see anything particularly useful in their analysis. It was originally intended to use the fracture surface of Core 2 as the “control sample” since the fracture was made during the core removal process. However, the lab created a fresh fracture surface in a portion of Core 5 instead for the “fresh vs. existing” comparison. Therefore, Core 2 was essentially unused in the examination.

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 12/7/2009

Status: Closed

Date Closed: 1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number:	82		
Individual Contacted:	Garry Miller	**Date Contacted:**	12/2/2009
Requestor/Inspector:	Dan Naus	**Category:**	Question

Request: According to MacTec petrographic report dated November 11, 2009 from MacTec, one-half of sample 21270 (Core #2) was sent to CTL for petrographic examination. In the CTL report dated November 2, 2009 there does not appear to be any reference to this sample. Were petrographic examinations performed on this sample, and if so, what are the results and where is it documented?

References:

Response Assigned to:	Charles Williams	**Date Due to Inspector:**	

Response:

The MACTEC report does say that half of core #2 was sent to CTL, but it does not specify what tests are to be performed on it. CTL has this half of Core #2, but has not done any testing on it at this time. This sample was taken from an area that did not contain delamination. There are currently no plans to perform any tests on this sample.

Misc Notes:

Response By:	Craig Miller		
Reviewed By:		**Date Response Provided:**	12/7/2009
Status:	Closed	**Date Closed:**	1/28/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 83

Individual Contacted: Garry Miller

Date Contacted: 12/2/2009

Requestor/Inspector: Dan Naus / George Thomas

Category: Information Request

Request: Describe what confirmatory NDE would be performed, after detensioning of additional tendons, in the areas that did not show any delamination in order to verify that the delamination has not propagated any further due to additional detensioning.

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

On completion of detensioning, Impulse Response (IR) testing will be performed over a representative number of previously examined panels to check for changes in average mobility values. In addition, areas indicated as being exposed to the maximum stress within a bay area will be examined. This population of panels provides an adequate validation that a delamination did not occur in bays other than 34. Boroscope inspections of core locations outside of bay 34 (including the dome) will be performed to confirm that a crack has not occurred in the core bores previously examined as part of the condition assessment. If there are multiple cores in a panel (~10' x 20' area outlined with a feature strip as shown on drawing 421-031), one core will be selected for inspection. Inspections will not be performed in cores that have been made inaccessible due to strain gage and reactor building temperature monitoring equipment installed. There is ample number of core bores available for inspection; therefore, this small population of cores is not a concern. (Ref. EC 75218, Section E)

Misc Notes:

Response By: Martin Souther

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 84

Individual Contacted: Charles Williams

Date Contacted: 1/12/2010

Requestor/Inspector: George Thomas

Category: Question

Request: With reference to the evidence sheets that refuted the root cause failure mode 8.4 - Inadequate Concrete Structure Monitoring/Maintenance (IWL), the SIT has the following observations:

1. The scope and description of this failure mode is unclear and not accurate. The monitoring of the concrete structure under the Containment ISI Program in accordance with ASME Section XI, Subsection IWL, in fact includes examination of concrete surfaces and the unbonded post-tensioning system (tendon surveillance) of the Class CC containment. Program referenced does not seem accurate.

2.The inadequacy of a CISI program in accordance with ASME Section XI, Subsection IWL is an issue of regulatory and procedural compliance. It is not by itself a failure mode. The program may help detect early signs of a degradation or potential failure mode or a failure mode after it has occurred. The implementation of the program may be inadequate if early signs of degradation identified during inservice inspection were not properly addressed and the degradation progressed into failure. Clarify what failure mode is being addressed by FM 8.4 represent?

3. The "Data to be collected" is incomplete/inadequate since (i) it does not look at past IWL inspection results of the concrete surfaces; (ii) it does not look at past IWL tendon surveillance reports. Documents referenced are not accurate.

4.The refuting and supporting evidence is incomplete/inadequate because: (i) Exhibit 2 is in fact results of visual examination of concrete surfaces between buttresses 3 and 4 performed during RF 16 after the delamination was discovered, and not "conducted a few days prior to beginning the SGR hole cut activities (FM 8.4 Exhibit 2…) as stated in the evidence sheet. None of the reports from past IWL inspections of the concrete surfaces were reviewed as evidence. (ii) The CR-3 containment has had a history of a significant number of hoop tendons, including some that go through the SGR Opening, not meeting the IWL acceptance by examination criteria during the recent three surveillances (i.e. Surveillances 6, 7 & 8). This could provide supporting evidence for the delamination root cause. None of the tendon surveillance reports from past surveillance were reviewed as evidence.

5. Further, the results of tendon surveillances 6, 7 & 8 were accepted by engineering evaluation. The cause (could be physical or calculation of predicted forces or both) of the lift-off forces of a large number of hoop tendons sampled (including extended sampling) not meeting the IWL acceptance by examination criteria was not adequately addressed and eliminated/corrected in the CR-3 tendon surveillance program. Is the cause of the larger than anticipated losses of prestressing force in several hoop tendons being addressed as part of the root cause assessment and, if so, where is it addressed?

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

1. The scope and description of Failure Mode 8.4 was revised to clarify.
2.The scope and description of Failure Mode 8.4 was revised to clarify it is addressing the concrete surface inspection only. The concrete-tendon-liner interactions are addressed in FM 6 series.
3.Similar comments received from reviewers. Revision to FM 8.4 clarified what is being reviewed and included copies of these documents.
4.Similar comments received from reviewers. Corrections made to the exhibits and descriptions. Scope was limited to concrete surface examinations per revised scope noted in items 1 and 2 above.
5.Tendon surveillance results are not part of this FM. The concrete-tendon-liner interactions are addressed in the FM 6 series.
Latest version of FM 8.4 provided to G Thomas on 4/3/10.

Misc Notes:

Response By: Craig Miller

Reviewed By:

Date Response Provided: 4/5/2010

Status: Closed

Date Closed: 6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 85

Individual Contacted: Charles Williams **Date Contacted:** 1/12/2010

Requestor/Inspector: George Thomas **Category:** Question

Request: With reference to evidence sheets for root cause Failure Mode 3.2, the description and exhibit photographs indicate presence of a secondary delamination near the liner. What is the location and extent of the secondary delamination? Is it being investigated in the root cause analysis?

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

There was evidence of a crack in the SGR opening near the liner after the liner was re-welded in place and the SGR lift system steel was removed. NCR 370853 was written to investigate and determine corrective actions. This assessment was documented in EC 74801Attachment Z16. See conclusion below.

Conclusion and Recommendations
The cracking along the stiffeners at the top and bottom walls of the SGR opening were not a result of the original delamination area. Due to the depth and location, these isolated, shallow cracks were most likely a result of heat stresses caused from the welding process during the reinstallation of the liner plate.
Due to the limited depth of the cracks, it is recommended to channel around any additional surface cracking to remove the cracked area prior to placement of the new concrete.
For the top wall cracks, there are two additional recommendations: 1) chip to the outer face of the existing concrete such that there is a slight angle to the channel made which will allow for concrete to be poured into the voided channel area that was created, or 2) have the pouring forms set such that when pouring concrete there will be enough hydraulic head to allow the concrete to fill the voided area of the channel. With both recommendations, one could be used in lieu of the other.

After detensioning for the delamination repair, inspection of the SGR opening revealed additional delamination at the same location as above, in the SGR opening near the liner plate. NCR 391514 was written to investigate and determine corrective actions. This secondary delamination extended approximately the width of the SGR opening, approx 7 feet above and 7 feet below the original SGR opening. Personnel involved with the repair project performed this investigation separately from NCR 358724, the Containment Root Cause Investigation. See NCR 391514 investigation below.

Action Request Number: 391514 Investigators: Ron Knott / Martin E. Souther

1. Adverse Condition Description

SGT NCR 30061-1015 DOCUMENTS THAT DURING INSPECTIONS REQUESTED BY PGN TO CONFIRM PREDICTED WALL CRACKS, ADDITIONAL CIRCUMFERENTIAL CRACKS WERE DISCOVERED AT THE TOP AND BOTTOM OF THE SGR OPENING. EC 75218 R7 SECTION B00 DESIGN HAD PREDICTED RADIAL CRACKS ABOVE AND BELOW THE SGR OPENING. ENGINEERING TO EVALUATE. THE SGT NCR IS IN THE FOLDER.

2. Investigation Summary

The containment wall concrete should not contain defects internal to the concrete structure. A defect has been detected at the bottom and top of the SGR opening. A circumferential separation has occurred in the concrete approximately three to four inches from the liner plate spanning between the sides of the SGR opening. Vertical stiffener angles, L3”x2”x ¼”, are welded to the liner plate. The three inch leg of the angle extends out from the liner plate with the two inch leg parallel to the liner plate. The circumferential separation at the bottom and top of the SGR opening will be discussed in this investigation as a single separation unless there is a distinct difference that requires explanation. The separation migrates closer to the liner plate as it approaches the sides of the SGR opening, appears to close, and is no longer visible. Boroscope inspections and wire probing were performed in an attempt to determine the extent of the separation away from the opening. The separation was found to extend in excess of four feet away from the opening.

Cracks had been previously identified at the bottom of the SGR opening near the liner plate and stiffener angles and were addressed in NCR 370853 and EC 75220 (Draft), Reactor Building Delamination Repair Phase 4 Concrete Placement. As part of the Containment Delamination Condition Assessment documented in EC 74801, Containment Structure- Extent-Of-Condition Core Bores and Condition Assessment Findings, the cracks were excavated to better characterize the condition. Hand tools were used to chip concrete along the length of the crack. The chipping revealed that the crack appeared to end within about two inches from the surface of the bottom opening. By comparison to the circumferential separation, it is assumed (after the fact) that the crack was still present.

Since SGR completion and liner restoration, the RB has been further detensioned in preparation for restoration efforts. Additional hoop tendons (138) were detensioned that span from Elevation 151’-3” to 239’-3”. Additional vertical tendons were also detensioned (54). Detensioning of the hoop tendons causes outward movement of concrete and liner to their neutral state. Two issues significantly influence the residual stresses at the detensioned condition. First, the liner section was removed after SGR detensioning (Stage 1). Prestress compression that existed in this zone was removed. After replacement of the liner section, further detensioning caused the structure to expand and generated tension in the liner. This tension is demonstrated on the attached plot from the ANSYS model (see NCR folder) used for restoration analysis of the Reactor Building. The ANSYS model reflects accurate bookkeeping of the stress state developed due to steam generator replacement activities. As indicated on the plot of hoop stress, tension exists in the liner to a distance of about 9 or 10 feet away from the edge of the SGR opening. Since the effect of tension on a curved surface is to flatten the surface, a significant pulling of the liner away from the inner surface of the concrete would be expected due to catenary action. At the sides of the SGR opening, hoop stresses dissipate very quickly. The tensile stress is neutral within a foot or so of the side of the opening. Secondly, the equipment hatch is stiff. Tensioned tendons in the equipment hatch keep the whole equipment hatch from moving outward. Effectively, the stiffened part of the equipment hatch will react as a unit. There are two stiffness transitions in this area. The first one is due to the change in thickness of concrete. The second is due to the change of reinforcing steel around the equipment hatch. The effect of reinforcing steel is small. However, inspection reveals that the secondary delamination stops at the transition where reinforcing steel begins. These cracks are not observed to the sides of the SGR opening because of the direction of curvature of the structure. Flattening of the liner can occur over the central part of the replaced liner to relieve the stresses in the liner.

Crack initiation due to heat effects and cold work associated with re-welding/replacement of the liner plate initiated a circumferential crack at the stiffener location. These effects are typically ignored in structural analysis. Further detensioning of the structure with a portion of the liner in a state of tension caused growth of the crack to a point near the neutral stress location of the liner. This point also coincides for the bottom of the SGR opening with the point where the equipment hatch stiffness minimized the effect of detensioning.

Engineering calculations had indicated the potential for radial cracking below and above the SGR opening and the possibility of such cracking was anticipated in EC 75218. Grout repair was considered to be a simple and viable solution at that time. Due to pending/expected repairs for Bay 34 around the SGR opening, other failure mechanisms that require the same repair where not investigated. As indicated above, the potential for additional stresses near the opening and potential cracking was recognized by engineering. This was included in the EC, but the latter point about other potential failure mechanisms, however, was not clearly communicated to construction.

At the time of the writing of this investigation, EC 75219 is implementing the concrete removal. The EC was revised to address removal of the concrete below and above the SGR opening due to the presence of the separation as an interim action to address this condition. The separation below the SGR opening has been totally removed by excavating down approximately seven feet from the bottom of the SGR opening and to the liner plate. The upper separation has also been excavated.

In preparation for detensioning and subsequent repair of containment, design engineering predicted radial cracking due to tendon detensioning. Caution was included in the EC to expect these cracks. Design engineering did not predict cracks that were initiated by alignment and re-welding of the liner. Better communication is needed between construction and the design engineering team regarding expected construction loading (cold work). Even though not typically required, engineering should describe effects associated with construction activities (cold working). Likewise, engineering should be more thorough with communication of effects of non-code loading conditions such as residual stresses (heat effects of welding).

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/4/2010

Status: Closed

Date Closed: 8/9/2010

Request Number: 86

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation 0102-0135-02 "Concrete Modulus of Elasticity and Specified Compressive Strength." There is no indication in the calculation that the variation in concrete modulus of elasticity for old and new concrete has been studied. Was there a parametric or sensitivity study performed? Please provide documentation that would justify that the values of modulus of elasticity recommended in this calculation for old and new concrete for use in the design basis calculations are conservative.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Calculation S09-0056 (MPR Calculation 0102-0135-02), Concrete Modulus of Elasticity and Specified Compressive Strength (Reference 1) established the concrete modulus of elasticity (E) for both the old and new concrete for Design Basis Calculations on their 28 day specified compressive strengths of 5000 and 7000 psi respectively and ACI 318-63, Section 1102(a) formula, as follows:

Eold = 4.03 x 106 and Enew = 5.12 x 106 psi

MPR Calculation 0102-0135-13, Concrete Elastic Modulus Sensitivity Analysis (Reference 2) performed a sensitivity study to investigate the effect of the concrete modulus of elasticity on the containment stresses under design basis load combinations. This parametric study was based on varying the individual moduli by 20%. Three scenarios are analyzed. For the first case, the elastic modulus of the new (stiffer) concrete is unchanged and the elastic modulus of the original (less stiff) concrete is decreased by 20% from those in Calculation S10-0012, Stresses Around the SGR Opening due to Design Basis Load Cases (Reference 5). In the second case, the elastic modulus of the new concrete is increased by 20% and the elastic modulus of the original concrete is unchanged. Both of these cases are used to examine the effect of large differences in elastic moduli on the stress results. The third case uses the same elastic modulus for both new and old concrete equal to the elastic modulus of the original concrete from Reference 5. A case of the new concrete being less stiff than the original concrete, or the old concrete having a higher modulus than 4.03 x 106 psi is not evaluated since the results of elastic modulus testing do not support the possibility of this occurring (from the Modulus of Elasticity and Poisson's Ratio tables from Reference 7 and the hardened properties table from Reference 8 for the old and new concrete, respectively).

Baseline Case (Values used in Design Basis calculations):

Eold= 4.03x106 and Enew = 5.12x106 psi. These baseline values are hereafter referred to as Enominal.

Case 1: Eold = 3.22 x 106 and Enew = 5.12 x 106 psi

Case 2: Eold = 4.03 x 106 and Enew = 6.14 x 106 psi

Case 3: Eold = 4.03 x 106 and Enew = 4.03 x 106 psi

The analyzed load combinations for the modulus of elasticity sensitivity study are:

• Load combination #1: 0.95D + Fa.eol + 1.5P + Ta (maximum tension)
• Load combination #2: 1.05D + Fo.rts – Eh – Ev (maximum compression)
Note: These load cases produced the maximum tensile and compressive stresses in Reference 5.

Where:
D = Dead load
Fa = Prestress under accident conditions (at end of life or return to service conditions denoted by “eol” and “rts” subscripts, respectively).
Fo = Prestress under normal conditions (at end of extended life or return to service conditions denoted by “eol” and “rts” subscripts, respectively).
Eh = Operating basis earthquake horizontal acceleration
Ev = Operating basis earthquake vertical acceleration
P = Accident pressure
Ta = Accident temperature

Reference 2 concluded that for Case 3 there were regions where the stress was more tensile/compressive than the same region of the Enominal configuration, but these stresses were less than those reported for Case 1 and 2 in these regions. For Case 1 and Case 2 configurations, the maximum increase in tensile or compressive stress was found to occur in or adjacent to the location of maximum stress. Since Case 1 and Case 2 are most limiting, only these two cases are further evaluated for their effects on the reinforcement design for Bay 34 in Calculation 32-9143357.

Calculation 32-9143357, Effects of the Modulus of Elasticity Variation on the Rebar Design for CR3 Containment (Reference 4), averaged the resulting stresses from Reference 2 for Cases 1 and 2 in a manner similar to the methodology described in Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall (Reference 3).

The average shear forces computed in Reference 4 were compared to the average shear forces reported in Reference 3 with the conclusion that the variation of the concrete modulus has a negligible effect on shear.

Reference 4 evaluated the reinforcement arrangement, designed in Reference 3, for the axial force and bending moment values resulting from the two load combinations for Case 1 and Case 2 moduli. Load Combination 1 is a factored load case and was evaluated by the Ultimate Strength Design method. Load Combination 2 is a normal operating load combination and was evaluated by the Working Stress Method.

Reference 4 evaluated the effects of operating and accident temperature gradient using the same approach as was used in Reference 3, i.e. using the methodology recommended in ACI 505-54, Specification for the Design and Construction of Reinforced Concrete Chimneys. The resulting thermal moments were then added to the primary moments.

Reference 4 concluded that the reinforcement design provided for Bay 34 is also adequate for the stresses resulting from the variation in the modulus of elasticity for the new and old concrete, i.e. Case 1 and Case 2 moduli.

Summary:
A parametric study was performed based on increasing or decreasing the value of E for the new concrete and decreasing the value of E for the old concrete from the nominal E’s that were calculated in Reference 1, and used for the reinforcement design for Bay 34. The reinforcement arrangement, designed in Reference 3, was then evaluated in Reference 4 to ensure it was adequate for the stresses reported from the parametric study. Reference 4 concluded that the reinforcement design for Bay 34 was adequate for all variations of E evaluated in the parametric study.

The nominal moduli of Eold= 4.03x106 and Enew = 5.12x106 psi that were established in Reference 1 as the base values for all design basis calculations has been shown to provide a conservative reinforcement design for

Bay 34 that envelopes the stress results obtained from the moduli sensitivity study.

References: (see request folder for references)

1. Calculation S09-0056, Concrete Modulus of Elasticity and Specified Compressive Strength
2. Calculation 0102-0135-13, Concrete Elastic Modulus Sensitivity Analysis (to be included in S10-0051)
3. Calculation S10-0030, Reinforcing Design for Delaminated Containment Wall
4. Calculation 32-9143357-000, Effects of the Modulus of Elasticity Variation on the Rebar Design for CR3 Containment (to be included in S10-0051)
5. Calculation S10-0012, Stresses Around the SGR Opening due to Design Basis Load Cases
6. Calculation S10-0051, Concrete Elastic Modulus Sensitivity Study (not issued). This calculation includes Reference 2 and Reference 4 calculations.
7. S&ME Transmittal 09-208-03 to Progress Energy Florida, November 16, 2009.
8. S&ME Report, "Phase III Test Report Mix Acceptance Testing for Crystal River Unit 3 Steam Generator Replacement Project, S&ME Project Number 1439-08-208, Contract 373812," June 19, 2009.

Attachments:

1. S&ME Transmittal 09-208-03 to Progress Energy Florida, November 16, 2009.
2. S&ME Report, "Phase III Test Report Mix Acceptance Testing for Crystal River Unit 3 Steam Generator Replacement Project, S&ME Project Number 1439-08-208, Contract 373812," June 19, 2009.

Misc Notes:

Response By:	Don Dykstershouse		
Reviewed By:		**Date Response Provided:**	11/16/2010
Status:	Closed	**Date Closed:**	12/6/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 87

Individual Contacted: Paul Fagan **Date Contacted:** 2/6/2010

Requestor/Inspector: George Thomas **Category:** Information Request

Request: Calculation 0102-0135-03, "Tendon Tension Calculation," the design input source cited for calculation of prestress losses is Reference 7 (Calc S-95-0082, Rev 3, 6th Tendon Surveillance - Generation of Tendon Force Curves). CR3 tendon surveillance results from the 6th, 7th, and 8th surveillances indicate that a significantly large number (~60% of the expanded sample in each surveillance) of hoop tendons did not meet the IWL acceptance by examination criteria of 95% of the predicted values determined in calculation S-95-0082. The cause was not addressed in the CR3 tendon surveillance program. The cause could be calculation, physical (excessive creep, shrinkage, steel relaxation than estimated) or both. Therefore, use of predicted losses based on calculation S-95-0082 as design input is questionable. Justify the use of calculation S-95-0082 as design input for prestress losses used for the containment delamination design basis calculations.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

Calculation S95-0082, 6th Tendon Surveillance – Generation of Tendon Force Curves, has been reviewed and it has been concluded that calculation is not consistent with several of the surveillance tests that have occurred over the past 10 years. NCR 379656 was initiated to resolve this discrepancy. One of the items that was reviewed was the value of creep. A 30 year specific value of 0.2 was used in the calculation; but a basis for the use of that value was not provided. Based on creep tests that were performed during the original construction and that were performed at Turkey Point Nuclear Power Plant, a different creep value has been determined and is in the range of 0.40. Use of this value makes the predictions more consistent with the results of the surveillance tests. Once this value is finalized, the S95-0082 calculation will be revised.

Tendon End-of-life Forces:
The tendon tensioning calculation (0102-0135-03) is not used in the Detensioning Analysis Calculation (0102-0135-06). Tendon forces used in return- to- service and end-of –life structural evaluations are those computed in Calculation S10-0028, Tendons/Forecast End of Life Force, and are summarized below.

• Dome Tendons and Undisturbed Hoop Tendons
Log-linear force / time trends are calculated using tendon lift-off forces measured during all surveillances completed to date. Return-to-service and end-of-life group mean forces are those defined by the trend lines.

• Vertical Tendons and Re-Tensioned Hoop Tendons
Return-to-service group mean forces are computed as minimum specified lock-off forces (70% GUTS) less mean elastic shortening losses.

End-of-life group mean forces are computed as return-to-service group mean forces less time dependent losses due to concrete shrinkage, concrete creep and pre-stressing steel relaxation.

Tendon forces during Detensioning:
The tendon forces being used to determine stress in the detensioned condition during the repair of containment are based on a regression analysis. These tendon force values have been determined in Calculation S10-0026, Tendon Regression Analysis for Detensioning Only.

References: (see request folder for references)
1. S10-0026, Tendon Regression Analysis for Detensioning Only
2. S10-0028, Tendons/Forecast End of Life Force

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	9/14/2010
Status:	Closed	**Date Closed:**	10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 88

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Question

Request: How does the expected tendon tension forces calculated in Calculation 0102-0135-03 compare with those obtained from the regression analysis of measured as-found lift-off forces of individual tendons from the past 8 surveillances of the CR3 containment for each type of tendon?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

For the current tendon tension forces (approximately 33 years after tensioning), the following comparison is provided:

Tendon TypePer Calculation 0102-0135-003Per 8th Tendon Surveillance Report
Horizontal1398 kips1369 kips
Vertical1474 kips1521 kips
Dome1376 kips1379 kips

These numbers are provided for comparison.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 5/10/2010

Request Number: 89

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: In Calculation 0102-0135-03, the prestress losses and tendon forces are calculated from date of initial structural integrity test (SIT), which for CR3 is not same as the date of initial tensioning. These tendon calculations should be based on the date of initial tensioning and not the initial SIT date.

Follow-up question:

Ensure that consideration is being given to the age of tendons when performing regression analysis. Use of the SIT date may change predictions.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The previous response is being replaced entirely with the response below (11/3/10).

Tendon forces computed in Calculation 0102-0135-03, Tendon Tension Calculation, were superseded and not used in subsequent design basis calculations. De-Tensioning Calculation 0102-0135-06, Tendon Detensioning Calculation uses the T = 33.3 years (after the SIT) tendon forces developed in Calculation S10-0026, Tendon Regression Analysis for Detensioning Only. These forces were determined by a regression analysis of cumulative tendon surveillance results.

Calculation S10-0012 (MPR Calculation 0102-0135-09), Stresses around the SGR Opening due to Design Basis Load Cases, which computes stresses around the SGR opening for the design basis load cases, uses minimum required pre-stressing forces documented in the Containment DBD (end of life dome tendon forces only) and forecast tendon forces determined in Calculation S10-0028, Tendons/Forecast End of Life Force. This calculation uses a regression analysis of cumulative surveillance results to forecast forces in undisturbed tendons and elastic shortening / time dependent loss computations to determine expected forces in re-tensioned tendons.

Consistent with prior analysis for CR3, the zero time origin for regression analysis and predictions was selected at the pre-operational SIT date. Other choices are available due to lack of specific guidance or requirements for trending. Valid selections include the following:

- Average date of tensioning.
- One day before the first surveillance.
- One year before the first surveillance (typically consistent with the SIT).

The average date of tensioning is an arbitrary point of reference for tendon surveillance data. This date is different for each group of tendons. The additional complication created by using this date was considered to not be warranted.

A study was performed to assess the impact of using the pre-operational SIT date as the zero time origin. In one case, regression analysis was performed with a data set that uses the average surveillance date and the SIT date to determine the time value. This method is referred to as the time-based approach. In another case, regression analysis was performed with a data set that includes individual ages for each tendon, where age is the elapsed time between the initial tendon tensioning date for an individual tendon and the date the tendon lift-off was performed for that tendon. This method is referred to as the age-based approach.

The result was a 1.2% change in the tendon force predicted at the end of extended license (60 years). Note: Additional lift-off measurements were made for the root cause analysis and containment repair efforts. Incorporation of these data reduce the difference (i.e. Result < 1.0 % change). There was essentially no effect for the return to service tendon force prediction.

Use of the surveillance date (one day prior) as the zero time origin for trending would have a similar effect in the opposite direction than described above. Therefore forecasts associated with regression analysis will continue to use the pre-operational SIT date as the zero time origin. This conclusion is specific to CR3 tendon tensioning data.

Calculation References: (see request folder for references)

S10-0001 (MPR 0102-0135-03) Tendon Tension Calculation
S10-0004 (MPR-0102-0135-06) Tendon Detensioning Calculation
S10-0012 (MPR 0102-0135-09) Stresses around the SGR Opening due to Design Basis Load Cases
S10-0026 Tendon Regression Analysis for Detensioning Only
S10-0028 Tendons/Forecast End of Life Force

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/2/2010

Status: Closed

Date Closed: 11/10/2010

Request Number: 90

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation 0102-0135-03. Section 1.0 of the calculation state that: "The values of tendon tension calculated herein will be used in structural analyses of the containment for ages 33 years and 60 years after the SIT." There is no discussion of indication in the calculation of the current design basis minimum required tendon force (1149 k, 1252 k, 1215 k, respectively, for vertical, horizontal and dome tendons - Ref. page 14 of CR3 Design Basis Document). Therefore, the end-of-life condition is being evaluated based on the expected (predicted) tendon force rather than the minimum required tendon force. This appears to be outside the CR3 current design basis. Please discuss and justify.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The tendon tension values that will be used to validate design basis conditions for end- of-– life (60 years) conditions will be minimum required prestress values. Current values are listed in DBD 11 (1149k, 1252k, 1215 kip, respectively for verticals, horizontal and dome tendons – Ref. page 14 of CR 3 Design Basis Document). These current values were considered for the containment repair project and determined to require revision. The values used for the containment repair design basis analysis for end- of-life tendon forces are as follows:

•1500 kips for vertical tendons
•1300 kips for horizontal tendons not detensioned during the containment repair project
•1435 kips for horizontal tendons detensioned during the containment repair project
•1215 kips for dome tendons

NOTE: The 50.59 Evaluation for Containment Repair is addressing the (additional) question of whether revising minimum required prestress values results in changing or exceeding a design basis limit for a fission product barrier.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/13/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 91

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation 0102-0135-03. Assumption 3 in Section 3.2 "Verified Assumption" states that: "The replaced concrete in the patch and the outer portion of the delamination will not be prestressed until 5 days after pouring. This…" This, as-stated, seems to be an unverified assumption since the start of retensioning would always be based on the required minimum strength of concrete for retensioning being achieved and verified in the field (based on tests of samples from the pour) rather than a time after the concrete pour. Please address this concern.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The assumption in Section 3.2 is accurate in that the replaced concrete will not be prestressed until 5 days after placing the concrete. In addition, the concrete must meet minimum concrete strength requirements. The minimum concrete compressive strength will be validated by using concrete breaking strength. This will be stated in EC 75220, Concrete Placement.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 5/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 92

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Question

Request: In Calculation 0102-0135-03 (pages 4 & 5), three tendon groups have been considered for horizontal tendons (Detensioned & Retensioned passing thru SGR Opening; Detensioned & Retensioned not passing thru SGR Opening; and Unadjusted). Why is it that only two tendon groups have been considered for vertical tendons (for the tendons traversing the SGR opening area that were cut) considered in the calculation? Is steel relaxation for new tendons different than for the original tendons?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The introduction to Section 5.3 of calculation 0102-0135-03 identifies that all of the vertical tendons that are detensioned and retensioned are considered to either pass through or are very near the SGR opening and delamination. Since the stiffness of the new concrete could conceivably affect all of these tendons to some degree, all detensioned and retensioned tendons are given the same losses as tendons that pass directly through the SGR opening. Therefore, there are no detensioned or retensioned vertical tendons that do not pass through the SGR opening.

The replaced tendons are not treated any differently than the retensioned tendons that are not completely replaced. The steel relaxation for the tendons is the same, and any possible reduction in relaxation for tendons that are reused instead of replaced is not accounted for.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/2/2010

Status: Closed

Date Closed: 10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 93

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation 0102-0135-04, "Finite Element Model Description," Section 5.0 related to Model Benchmarking Results. The benchmarking or validation of the finite element model is based on just one load case example. One example does not adequately validate a complex finite element model such as that developed for the CR3 containment analysis that uses multiple program features, loads and boundary conditions. Provide and document additional examples that provide a comprehensive validation of the finite element model used. Examples to consider may include comparison to results from original analysis documented in the FSAR, results of response measurements from original structural integrity test, etc.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Complete verification of the model geometry and boundary conditions was performed separate from the benchmark discussed in Section 5.0. The comparison to a known solution in Section 5.0 is confirmatory in nature and in no way was intended to be a substitute for rigorous model development and verification. The validation of the model is enhanced by 3 dimensional computer graphical examinations demonstrated in the calculation. It should be noted that the figures displayed in Calculation S10-0002 (MPR Calculation 0102-0135-04) are only samples of graphical checks made possible through post processing applications.

As an additional check, a benchmark of the ANSYS model results compared to SIT predicted results has been made in Calculation S10-0040. (MPR calculation 0102-0135-12) Comparison of ANSYS Results to Kalnins SIT Calculation and SIT Measurements provided acceptable benchmarking results.

Reference:
Calculation S10-0040, Comparison of ANSYS Results to Kalnins SIT Calculation and SIT Measurements

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/24/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

Request Number:	94		
Individual Contacted:	Paul Fagan	**Date Contacted:**	2/6/2010
Requestor/Inspector:	George Thomas	**Category:**	Information Request

Request: Refer to Calculation 0102-0135-04. Provide a discussion in the calculation with regard to the objectivity and adequacy of the finite element mesh used in the model.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

The element mesh size is verified in Calculation S10-0014, "Finite Element Model Benchmarking." Calculation S10-0014 compares Kalnins program results to the same model evaluated with ANSYS using a mesh size similar to containment analyses. An additional check in the calculation doubles the number of elements in the ANSYS finite element model. The maximum displacement, membrane and membrane plus bending stresses with a finer mesh were within 1% of the results with the coarser model. This effect is considered to be trivial. Therefore, the mesh size used is adequate for structural analysis and design of reinforcing steel.

REFERENCE:
1. Calculation S10-0014, "Finite Element Model Benchmarking." (see request folder for reference)

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	10/5/2010
Status:	Closed	**Date Closed:**	10/19/2010

Request Number: 95

Individual Contacted: Paul Fagan **Date Contacted:** 2/6/2010

Requestor/Inspector: George Thomas **Category:** Information Request

Request: Referring to Calculation 0102-0135-04, the concrete thermal expansion coefficient has been considered as 4.25 x 10-6. Based on a review of ACI 349 and other text books, 5.5 x 10-6 has been recommended. Please provide justification that the value of thermal expansion coefficient used in this calculation is in compliance with the CR3 design basis.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

NOTE: This response also includes a review of the thermal expansion coefficient used in the design basis calculation S10-0012 (MPR calculation 0102-0135-06), "Stresses Around the SGR opening due to Design Basis Loads".

ACI 349-06, Appendix E.3.3 (d) states, the coefficient of thermal expansion (α) of concrete may be taken as 5.5 x10 -6 per degree Fahrenheit unless other values are substantiated by "tests".

Based on the ACI description above, allowance is provided in the ACI Code to use other values. It should be noted that the inclusion of a recommended value in ACI-349 does not constitute design basis since the Crystal River Unit #3 containment building was not designed to ACI-349.

Research was conducted to determine what the appropriate value of α should be for concrete made from limestone aggregate and arrived at α = 4.25 x 10-6 /degrees Fahrenheit. This value was based on testing referenced in Table 2.2.38 of the "National Cooperative Highway Research Program, Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures," March 2004. The value selected was based on the average of the values listed for concrete made from limestone aggregate. This is the most accurate value for thermal expansion based on the material properties.

A search of the CR3 FSAR, including the PSAR Supplement #6 (Analysis and Design of Tendon Anchorage Zones) and the Design Basis Document for Containment, DBD 011, has been completed and no defined value for the coefficient of thermal expansion was found.

The coefficient of thermal expansion (α) is utilized in three calculations:

1. Calculation S10-0004 (MPR calculation 0102-0135-06), "Tendon Detensioning Calculation". One of the requirements for this calculation was to predict the extent of cracking in the unreinforced wall (after the delaminated concrete was removed) that could be reasonably expected as a result of detensioning vertical and hoop tendons. To achieve realistic results, the calculation used α = 4.25 x 10-6, the most accurate value as discussed above, and considered a 10 degree gradient through the containment wall.

2. Calculation S10-0012 (MPR calculation 0102-0135-09), "Stresses around the SGR Opening due to Design Basis Load Cases" is a design basis calculation that includes both normal and accident load combinations. However, these load combinations include only the thermal load due to the differential expansion of the liner and concrete (coefficient of expansion of the concrete has no relevance for this load), and the thermal axial load which results from an increase or decrease in the average wall temperature over the walls steady state temperature, i.e. {(outside temperature – inside temperature)/2 -70 degrees F, where 70o is the concrete

placement temperature per the FSAR} and is based on α = 4.25 x 10-6. Per calculation S10-0002 (MPR calculation 0102-0135-04), “Finite Element Model Description”, the minimum average wall temperature input to the ANSYS finite element model is 66 degrees F (inside temperature = 102o, outside temperature =30o). Therefore, although using a more realistic thermal coefficient is allowed by ACI 349, and neither the FSAR nor the original design basis calculations give any guidance on what value should be assigned, this value (α = 4.25 x 10-6) was only used in the thermal axial load case based on a decrease in the average wall temperature of -4o (70-66). The membrane compression/tension stress resulting from a 4o decrease in the shell temperature is relatively small based on unit stress ~ ∆T α E:

σ = 4 x 4.25 x 10e-6 x 4e6 = 68 psi
or
σ = 4 x 5.50 x 10e-6 x 4e6 = 88 psi (i.e. net increase in stress is +/-20 psi)

Note that bending stresses of any significance are not expected from an axial temperature increase/decrease in a structure.

3. The load due to the thermal gradient through the concrete wall is addressed separately in calculation S10-0030, Reinforcement Design for Delaminated Containment Wall, which conservatively assumed α = 5.5 x 10-6 per degree Fahrenheit.

Conclusions:
(i) Using a more realistic value for the coefficient of thermal expansion (α) based on test results is accepted by ACI 349.
(ii) ACI 349 is not part of the CR3 design basis for containment thermal analysis, however, the basis for identifying a realistic value should include a review of all relevant documentation, including ACI.
(iii) Neither the FSAR, PSAR Supplements or the DBD for Containment contain a value for the coefficient of thermal expansion.
(iv) Calculation S10-0004 (MPR calculation 0102-0135-06), Tendon Detensioning Calculation uses the more realistic value of α = 4.25 x 10-6 to help in determining the amount of concrete cracking that could be expected as a result of detensioning.
(v) Calculation S10-0012 (MPR calculation 0102-0135-09), Stresses around the SGR Opening due to Design Basis Load Cases, is a design basis calculation, however, even though using a value of α = 4.25 x 10-6 is acceptable (based on (i) thru (iii)), only the axial thermal load case used this value and the resulting membrane stresses due to a decrease of 4o F in the shell temperature is insignificant.
(vi) The thermal gradient through the concrete wall is evaluated separately in calculation S10-0030, Reinforcement Design for Delaminated Containment Wall, which conservatively assumed α = 5.5 x 10-6 per degree Fahrenheit.

References:

1.Calculation S10-0004 (MPR calculation 0102-0135-06), Tendon Detensioning Calculation
2.Calculation S10-0012 (MPR calculation 0102-0135-09), Stresses around the SGR Opening due to Design Basis Load Cases
3.Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/24/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 96

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation 0102-0135-05 , "Conduit Local Stress Analysis." This calculation, based on an axis-symmetric computer model, does not consider the bending moments and stress gradients that results from detensioning tendons, nor does it consider the eccentricity of the vertical tendon load. These effects could produce more limiting tensile stresses around the conduits and could invalidate the conclusion of the calculation. Please address the above effects on the conclusion for the calculation and how does it compare to the computer simulations being performed by PII, as part of the root cause analysis.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Calculation S10-0003, “Conduit Local Stress Analysis”, (0102-0135-05) has not been used in any design basis calculations. The calculation will be revised to prevent use in any future calculations.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/5/2010

Status: Closed

Date Closed: 10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 97

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Question

Request: Calculation 0102-0135-05 is based on tendon conduit thickness of 1/16". What is the effect of a 1/8" thick conduit on the analysis that is also used on the CR3 containment cylindrical wall?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Calculation S10-0003, "Conduit Local Stress Analysis", (0102-0135-05) has not been used in any design basis calculations. The calculation will be revised to prevent use in any future calculations.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/5/2010

Status: Closed

Date Closed: 10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 98

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Referring to Calculation 0102-0135-06 "Containment Detensioning Evaluation," please confirm that the cuts that have been made in the delaminated area have been considered in the finite element model. If these cuts were not modeled, please provide justification.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The concrete cuts that have been made in the delaminated area per EC 75000 have not been included in the model. The cuts that were made were only partial cuts to the delaminated are and were typically less than 8 inches in depth. The structure was monitored while the cuts were being made and the displacement was minimal. (close to zero) Therefore, new discontinuities in the existing structure were not introduced and the stiffness of the existing structure should not be affected. Since the cuts did not penetrate beyond the delaminated concrete, it was concluded that the cuts do not need to be modeled in finite element model.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 7/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 99

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Referring to Calculation 0102-0135-06, the retensioned structure has been evaluated for 0.95D + Fa + 1.5P + Ta design basis load condition. On page 67 of this calculation, it is concluded that the stresses in certain areas exceed the design basis acceptance criteria. No further information is provided in this calculation relative to how the design basis will be maintained. Please provide further discussion.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The detensioning calculation (0102-0135-06) analyzes and provides results of the finite element analysis. It is anticipated that disposition of some overstresses may be required. Since this calculation is primarily evaluating containment while the tendons are detensioned, any overstress will be evaluated in a subsequent calculation. EC 75218 describes the overstress and provides a general resolution of how those stresses will be dispositioned when the design basis calculation is completed.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 8/27/2010

20-Apr-11 7:56:36 AM

Request Number: 100

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Referring to Calculation 0102-0135-06, please confirm that the liner plate design temperature of 281 degrees F (CR3 FSAR, Page 45) has been considered in evaluating the retensioning structure for the design basis load condition of 0.95D + Fa + 1.5P +Ta.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The purpose of Reference 1 is to evaluate a tendon detensioning sequence that would be applied during the repair of the delaminated concrete. An important component of this evaluation is to make a preliminary investigation of the repaired (retensioned) containment for 60 year end-of-life forces to ensure that the repaired containment meets the design basis acceptance limits (Ref. 2, Section 5.2.3.3.1), for all applicable design basis load cases (Ref. 2, Table 5-3).

The liner plate is included in the ANSYS finite element model (FEM) as a single layer of four node elastic-plastic shell elements. For the design basis load case of 0.95D + Fa + 1.5P + Ta, the accident temperature applied to the steel liner plate elements is the design temperature of 281 degrees F (Ref. 1, Section 4.4.3 and Ref. 2, Section 5.2.5.2.2). Note that the reference temperature (original concrete placement temperature) is 70 degrees F (Reference 3), therefore, the actual increase in liner plate element temperature (producing a corresponding increase in liner strain) applied to the ANSYS FEM model is 281 – 70 = 211 degrees F.

References:
1.Progress Energy Calculation S10-0004 (MPR Calculation 0102-0135-06), Tendon Detensioning Calculation, Revision 1.
2.CR3 FSAR, Revision 32.
3.Design Basis Document for the Containment, Tab 1/1, Revision 7.

(see request folder for Calculation S10-0004)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/18/2010

Status: Closed

Date Closed: 8/27/2010

Request Number: 101

Individual Contacted: Paul Fagan **Date Contacted:** 2/6/2010

Requestor/Inspector: George Thomas **Category:** Information Request

Request: Referring to Calculation 0102-0135-06, please confirm that Figure 5-10 of CR3 FSAR has been considered in the evaluation of the retensioned structure for the design basis load condition of 0.95D + Fa + 1.5P + Ta.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

Calculation S10-0004 (0102-0135-06), "Tendon Detensioning Calculation", (Reference 1) performed a preliminary assessment of the stresses resulting from retensioning the tendons that had been detensioned during the repair process. The accident thermal load considered consisted of an accident temperature of 281^{0}F (Design Basis Document for Containment DBD 1/1, Reference 5) applied to the liner plate and an average (axial) temperature of 66^{0}F; based on Winter Normal Operating temperatures as tabulated in DBD 1/1 for inside and outside of the containment wall and buttresses. The design temperatures listed in DBD 1/1 are based on the Thermal Curves contained in the FSAR Figures 5-10 and 5-11.

The thermal gradient across the wall section for the accident load combination in Calculation S10-0004 (0102-0135-06) is shown in FSAR Figure 5-11 "Reactor Building ISO-Thermal Curves for Winter Normal Operating Temperature Conditions." The use of the normal operating temperature gradient at the beginning of the LOCA event is discussed further in the response to S.I.T. Question 119. The stresses were evaluated in EC 75218, Section B.6-10 under the heading "Design Basis Condition Assessment". The purpose of reporting these stresses in S10-0004 was to provide a reasonable assurance that by using additional analysis techniques such as increasing tendon prestress, adding reinforcement and/or refining the FEM model, that a final stress condition could be achieved that complied with the design basis acceptance criteria described in the FSAR (and ACI 318-63). It was recognized that the thermal gradient across the section would eventually be evaluated using the methodologies described in ACI 505-54. However, for the purpose of Calculation S10-0004, interaction diagrams were prepared with stresses from the finite element analysis for accident loading that included a normal operating thermal gradient.

Calculation S10-0012 "Stresses Around the SGR Opening due to Design Basis Loads" (Reference 2) determines the stresses in the repair area (Bay 34) due to design basis load combinations. The calculation considered a thermal accident load that consists of a 281^{0}F temperature applied to the liner plate and the minimum average wall temperature of 66^{0}F based on Winter Normal Operation temperatures for inside and outside of the containment wall and buttresses.

The thermal gradient across the wall was evaluated in S10-0030, "Reinforcement Design for Delaminated Containment Wall," per the requirements of ACI 505-54, as required by the FSAR. The peak accident thermal gradient does not exist on the containment wall simultaneously with the peak accident pressure due to the time lag between the two as explained in Section 4.4 of Reference 6. At peak pressure, the thermal gradient on the wall is lower than the peak accident thermal gradient. However, Calculation S10-0030 conservatively evaluates the accident thermal gradient based on the peak Winter Accident temperatures, as listed in DBD 1/1 of 158^{0}F for inside temperature and 37^{0}F for outside temperature. The resulting thermal moments across the shell were added to the moments derived from Calculation S10-0012 (based on peak pressure) and the resulting combined moments evaluated per the requirements of the FSAR and ACI 318-63 in Calculation S10-0030.

In summary, Figure 5-10 has been conservatively considered in design calculations for the repair in Bay 34; S10-0012 and S10-0030.

As described in Calculation S10-0004, further analysis is required to assess impact to other parts of the Reactor Building (Reference Calculation S10-0057). This calculation implements the original design approach where the normal operating thermal gradient is combined with peak accident pressure and peak liner expansion. See Reference 7 for additional discussion.

References: (see request folder for references)

1. Calculation S10-0004, Rev. 1, "Tendon Detensioning Calculation", (MPR Calculation 0102-0135-06).
2. Calculation S10-0012, Rev.1, "Stresses Around the SGR Opening due to Design Basis Loads," (MPR Calculation 0102-0135-09).
3. Calculation S10-0030, Rev. 1, "Reinforcement Design for Delaminated Containment Wall."
4. Calculation S10-0057, (not issued), "Steam Generator Replacement Construction Impact on Containment Stress." (MPR Calculation 0102-0135-26).
5. Design Basis Document for Containment DBD 1/1, Revision 8.
6. Crystal River Unit No. 3 – PSAR Supplement 6, "Analysis and Design of Tendons Anchorage Zone."
7. Response for S.I.T. Question 119.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/13/2010

Status: Closed

Date Closed: 1/10/2011

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 102

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Referring to Calculation 0102-0135-08, it is stated that the peak ground acceleration with a 1.5 multiplier factor was used for seismic evaluation. Please provide justification that the acceleration used in this calculation is conservative when compared to the containment building accelerations, at different mass points along the height of the structure, determined in the design basis seismic analysis.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Seismic response spectra for CR3 are contained in SP-5209, CR3 Seismic Qualification (Reference 1) for all major structures, including the containment shell. The mass model used to generate the floor response spectra (FRS) for the containment shell is shown on page 30. This “stick” model shows 8 mass points; FRS were generated for 5 of these mass points at Elevations 106’, 123’, 139’, 154’ and 270’. The maximum, non-amplified portion of the FRS, i.e. the zero period acceleration (ZPA), represents the maximum peak acceleration of the floor/mass at that level.

The Operating Basis Earthquake (OBE) horizontal seismic accelerations are obtained from Pages 35 to 44 of Reference 1. As previously noted, the ZPA of the response spectra provides the maximum OBE acceleration for the shell at the elevations of interest. OBE vertical seismic is 2/3 of the horizontal accelerations. The Safe Shutdown Earthquake (SSE) accelerations are twice the OBE accelerations.

Reference 1 and the FSAR do not contain the magnitude of the lumped masses applied at each of the 8 mass points in the original seismic analysis of the shell. Therefore, an estimate of the tributary mass of the containment acting at each of the 5 elevations for which FRS are available (as noted above) is calculated below based on information derived from S10-0006 (Reference 2).

Mass of the dome = 5.74E6 lbs
Mass of ring girder = 9.29E6 lbs
Mass of cylinder = 3.32E7 lbs (93’ to 250’)
Total mass = 4.82E7 lbs
SSE horizontal acceleration = 0.405g
SSE vertical acceleration = 0.27g

Mass Point #5: Referring to a sketch showing a cross section of containment locating the mass points by elevation (sketch in folder), mass point 5 consists of the dome + ring girder + 48’ of the cylinder wall = 5.74E6 + 9.29E6 + 3.32E7 x 48/157 = 2.518E7 lbs

Mass Point #4: 3.32E7 x (48+7.5)/157 = 1.174E7 lbs
Mass Point #3: 3.32E7 x (31/2)/157 = 3.278E6 lbs
Mass Point #2: 3.32E7 x (33/2)/157 = 3.490E6 lbs
Mass Point #1: 3.32E7 x (30/2)/157 = 3.172E6 lbs

Zero Period Acceleration from Reference 1:

(see request folder for table)

From Ref. 2:
Total mass of (cylinder + dome + ring girder) = 4.82E7 (Section 6.2). Note that the contribution of the mass from the last node point (#1) does not include the 6.5' height of wall from the top of the mat to the mid-point between node #1 and the mat. This is typical for response spectra analysis at a rigid support.
Lateral acceleration ah= 0.405g (Section 6.1)
Vertical acceleration av = 0.27g (Section 6.1)
 Total vertical seismic force = 0.27 x 4.82E7 = 1.3E7 lbs
Total horizontal seismic force = 0.405 x 4.82E7 = 1.952E7 lbs
Moment due to horizontal seismic = 2.11E9 ft.lbs

Compare Results from Ref. 2 to Results in Table 1:
Total vertical seismic force per Ref. 2 = 1.3E7 > 9.87E6 lbs Ref. 2 results are conservative

Total horizontal seismic force per Ref. 2 = 1.952E+07 > 1.48E+07 lbs Ref. 2 results are conservative

Total OTM at the base per Ref. 2 = 2.11E+09 > 1.93E+08 ft.lbs Ref. 2 results are conservative

Conclusion:
The seismic forces and overturning moments obtained from Reference 1 are more conservative than applying the accelerations at different mass points along the height of the building. Note that the mass points used in this evaluation and the corresponding floor response spectra are per CR3 Specification SP-5209 which reflects the current design basis for the plant.

References:
1.Specification SP-5209, Revision 0, CR3 Seismic Qualification
2.Calculation S10-0006, Revision 1, Seismic, Wind, and Tornado Evaluation and Delamination Depth Evaluation for Detensioned State (MPR Calc. 0102-0135-08)
3.Sketch showing Cross Section of Containment Locating the Mass Points by Elevation (located in folder)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/4/2010

Status: Closed

Date Closed: 11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 103

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Referring to calculation 0102-0135-08, the flexural, membrane and shear stresses due to tendon detensioning, which are additive to the dead load and seismic/tornado induced stresses, have not been considered. Please provide justification.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The purpose of this calculation is to verify that the containment shell will not fail catastrophically in the detensioned state due to natural phenomena, i.e. earthquake, wind and tornado. The calculation addresses the impact that the detensioned containment could possibly have on the adjacent fuel pools during these three events. To evaluate the potential impact, the structure has been conservatively evaluated for earthquake and tornado loads without taking credit for any prestress load that exists in the partially detensioned structure. This conservative assumption maximizes the (membrane) and (membrane + bending) tensile stresses that could result in concrete cracking and possible failure of the detensioned containment shell. Prestress forces cause radial shear and externally applied lateral loads generate in-plane shear. Additionally, shear stresses are shown to be very small. Add It is assumed that a fully detensioned containment shell essentially returns the shell to its original stress free condition (except for dead load) prior to tendon tensioning. This is the configuration that has been conservatively evaluated in calculation 0102-0135-08. (S10-0006)

REFERENCES:
S10-0006 R1 Seismic, Wind, and Tornado Evaluation and Delamination Depth Evaluation for Detensioned State

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/4/2010

Status: Closed

Date Closed: 10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 104

Individual Contacted: Paul Fagan

Date Contacted: 2/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: CR3 FSAR refers to ASCE Paper 3269 for determining drag coefficient. Referring to Calculation 0102-0135-08, please confirm that the drag coefficient used in this calculation is consistent with the CR3 FSAR.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The drag coefficient used in the calculation is 0.38 @ 106. A review of ASCE Paper 3269 results in a value of 0.35 @ 106. Thus, these values are consistent.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 5/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 105

Individual Contacted: Paul Fagan

Date Contacted: 2/10/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to Failure Mode 1.3

1. Under "Verified Supporting Evidence", Item a., the comparison of hoop prestress level of 978 k/ft to a maximum of 730 k/ft in Bechtel-built plants is misleading since they are comparisons between apples and oranges. The 978 k/ft and 1025 k/ft prestress forces calculated for CR3 in Exhibit 2 are based on maximum tendon force of 1635 k (based on maximum allowed lock-off stress of 0.7Fu, which is the maximum lock-off stress typically allowed (some plants may be up to 0.73Fu) in the tendon wires for all plants). The 730 k/ft force and the effective prestresses for other plants in Table 2-1 on page 20 of Exhibit 2 are in fact "minimum design" effective prestress levels for the plants listed. The minimum design prestress force for hoop tendons at CR3 is 1252 K/tendon (see CR3 DBD), which corresponds to an upper bound value of (1252 k) x 2/(38.25"/12) =786 k/ft (using the smallest spacing of tendons, i.e. 2 tendons over a height of 38.25", note that the spacing is generally higher than this along the wall). For consistency, based on number of tendons along the height of the containment wall, this prestress level would be 1252 k x 94/157' = 750 k/ft, compared to a maximum value of 730 k/ft in Table 2-1 of Exhibit 2.

2. The same comment as in request #105 applies to Exhibit 8. The values of 793 k/ft and 1325 k tabulated for CR3 are not based on minimum design effective presenters, whereas for the other plants the corresponding numbers are based on minimum design prestress levels. For CR3 the minimum design values for hoop tendons are 750 k/ft and 1252 k as calculated in request #105. Also, the "Design Force" and "effective Prestress" columns do not correspond to the same location across the wall thickness - the former is calculated at the inner radius of the containment whereas the latter is an average value at the middle of the containment wall thickness. For CR3, the corresponding design force at the center of the wall thickness would be 515 k/ft x 65'/66.8' = 501 k/ft. Therefore, the prestress design ratio is 750/501 = 1.5, which is consistent with the design basis calculations,. In general, the design of containments (including CR3) involve separate factored load combinations that include P, 1.25P and 1.5 P (ASME Section III, Div 2). Typically, the prestress levels for post-tensioned containments are designed based on the 1.25P or the 1.5P cases so that the concrete is not subjected to membrane tension even during the SIT at 1.15P or even if the estimated accident P is exceeded). CR3 prestress design is based on the 1.5P level (FSAR Section 5.2.3.2.1). Therefore, prestress design ratio values such as 1.6 are exaggerations.

3. Exhibit 1, at the top of page 3, states that the FSAR specified prestress level of 829 k/ft. Where in the FSAR is this specified?

4. Item 6 under "Data to be collected and Analyzed" on page 1 of the evidence sheet. Maximum tension allowed should be 80% of minimum ultimate strength (and not 80% of yield).

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

1.FM 1.3 and its Exhibit 1 have been revised to provide a better comparison based on reviewers comments.
2.Exhibit 8 was deleted and replaced with the excerpts from the original Gilbert calculation based on reviewers' comments.
3.Exhibit 1 now references the original Gilbert calculation specifying 829 kips/ft.
4.This FM now reviews for 80% of Guaranteed Ultimate Tensile Strength.
A similar question, NRC Request 127 was also addressed on FM 1.1.

The revised FM 1.3 was provided to G Thomas on 5/28/10

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 6/1/2010

Status: Closed

Date Closed: 6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number: 106

Individual Contacted: Paul Fagan

Date Contacted: 2/10/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: The same comment as in request #105 applies to Exhibit 8. The values of 793 k/ft and 1325 k tabulated of CR3 are not based on minimum design effective prestress, whereas of the other plants the corresponding numbers are based on minimum design prestress levels. For CR3 the minimum design values for hoop tendons are 750 k/ft and 1252 k as calculated in request #105. Also, the "Design Force" and "effective Prestress" columns do not correspond to the same location across the wall thickness - the former is calculated at the inner radius of the containment whereas the latter is an average value at the middle of the containment wall thickness. For CR3, the corresponding design force at the center of the wall thickness would be 515 k/ft x 65'/66.8' = 501 k/ft. Therefore, the prestress design ratio is 750/501 = 1.5, which is consistent with the design basis calculations. In general, the design of containments (including CR3) involve separate factored load combinations that include P, 1.25P and 1.5 P (ASME Section III, Div 2). Typically, the prestress levels for post-tensioned containments are designed based on the 1.25P or the 1.5P cases so that the concrete is not subjected to membrane tension even during the SIT at 1.15P or even if the estimated accident P is exceeded). CR3 prestress design is based on the 1.5P level (FSAR Section 5.2.3.2.1). Therefore, prestress design ratio values such as 1.6 are exaggerations.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Misc Notes: Question transferred into Request #105.

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 2/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:36 AM

Request Number:	107		
Individual Contacted:	Paul Fagan	**Date Contacted:**	2/10/2010
Requestor/Inspector:	George Thomas	**Category:**	Question

Request: Exhibit 1, at the top of page 3, states that the FSAR specified prestress level of 829 k/ft. Where in the FSAR is this specified?

References:

Response Assigned to:	Don Dyksterhouse	**Date Due to Inspector:**	

Response:

Misc Notes: Question transferred into Request #105.

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	
Status:	Closed	**Date Closed:**	2/25/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number:	108		
Individual Contacted:	Paul Fagan	**Date Contacted:**	2/10/2010
Requestor/Inspector:	George Thomas	**Category:**	Information Request

Request: Item 6 under "Data to be collected and Analyzed" on page 1 of the evidence sheet. Maximum tension allowed should be 80% of minimum ultimate strength (and not 80% of yield).

References:

Response Assigned to:	Don Dyksterhouse	**Date Due to Inspector:**	

Response:

Misc Notes: Question transferred into Request #105.

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	
Status:	Closed	**Date Closed:**	2/25/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 109

Individual Contacted: Garry Miller

Date Contacted: 2/11/2010

Requestor/Inspector: Anthony Masters

Category: Question

Request: After de-tensioning, what assessments or extent of condition reviews will take place to the containment structure to verify new condition resulting from the change? What is the extent (sampling, entire structure, etc.) timing (after de-tensioning but prior to concrete removal, during removal, etc..), and documentation of that process?

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

See response to Request Number 83 for extent of examination and inspection timing and scope. The results of the Impulse Response and boroscope inspections will be documented in the work package. (Ref. EC 75218, Section E)

Misc Notes:

Response By: Martin Souther

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 110

Individual Contacted: Garry Miller

Date Contacted: 2/11/2010

Requestor/Inspector: Anthony Masters

Category: Question

Request: How is the acoustic emission monitoring and condition assessment different than the impulse echo and IR scans for condition assessment and extent of original condition in regards to requirements and controls for NDE procedures, qualifications, quality assurance, and certifications?

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

Radial strain gauges and acoustic emission monitoring will be utilized to help prevent and mitigate the non-delaminated portions of the containment structure from having any damage created in these areas during the detensioning process. The monitoring is not credited for validating any design function. The monitoring is not an input to a design change. Vendors involved in performing the monitoring have provided detail information supporting the use and evaluation of data generated. The vendors monitoring methods have been accepted by Progress Energy. (Ref. EC 75218, Section E)

Misc Notes:

Response By: Martin Souther

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 111

Individual Contacted: Garry Miller

Date Contacted: 2/11/2010

Requestor/Inspector: Anthony Masters

Category: Question

Request: Will the monitoring and condition assessment plan be modified to include any concerns or issues that result from the de-tensioning sequence analysis and modeling? For example, the critical areas were defined and presented by engineering to acoustic emmision contractor, MISTRAS. The contractor then developed the monitoring plan, placement, arrangement to include tighter areas in two areas determined to be critical. With the on-going modeling of the various de-tensioning sequence options, it appears that one area in the panels between buttresses 1 and 6 continues to exemplify cracking issues. Would this be considered a critical area?

References:

Response Assigned to: Paul Fagan

Date Due to Inspector:

Response:

On completion of detensioning, Impulse Response (IR) testing will be performed over a representative number of previously examined panels to check for changes in average mobility values. In addition, areas indicated as being exposed to the maximum stress within a bay area will be examined. Section E.1.3 of EC 75218 includes the scope of the IR testing, which includes specific high stress areas identified by PII in Attachment Z40 of EC 75218.

Misc Notes:

Response By: Martin Souther

Reviewed By:

Date Response Provided: 3/21/2010

Status: Closed

Date Closed: 7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 112

Individual Contacted: Garry Miller

Date Contacted: 2/11/2010

Requestor/Inspector: Anthony Masters

Category: Question

Request: Prior to de-tensioning, will the models be developed to include the hatches or other areas of the structure that may affect the response?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

For the ANSYS model used for detensioning, the equipment hatch, buttresses, ring girder, basemat, and dome were modeled. The model for detensioning is described in Calculation S10-0004, Tendon Detensioning Calculation. This level of detail was determined to be adequate for detensioning with respect to the structural response of containment. In addition, similar models were used in conjunction with the containment detensioning analysis developed by PII, which included a greater level of detail in selected areas.

For the design basis calculation, a refined model of the equipment hatch has been developed in ANSYS to analyze the equipment hatch area with specific tendons draped around the equipment hatch opening. This analysis, including the equipment hatch modeling, is documented in Calculation S10-0015, Equipment Hatch Submodel (Refined Model).

Reference:
Calculation S10-0004, Tendon Detensioning Calculation
Calculation S10-0015, Equipment Hatch Submodel (Refined Model)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/18/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 113

Individual Contacted: Garry Miller

Date Contacted: 2/11/2010

Requestor/Inspector: Anthony Masters

Category: Question

Request: How is the increased steel liner exposure to the elements being addressed (i.e. the uncoated liner is being exposed to the weather and corrosive environment longer than originally planned in the steam generator replacement scope - is this a concern and how is it being assessed)?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The Containment IWL Repair Plan, Section 14.3 (Refer to EC 75220, Attachment Z08) requires that the exposed liner is examined to ensure its readiness for concrete placement. All debris, oil and deleterious material (which would include loose surface rust/corrosion) must be removed from the liner. The IWE/IWL RPE will be responsible (or designee) for evaluating the acceptability of any surface corrosion identified during the IWL inspection and its effect on the liner plate integrity. The requirement for inspecting the liner plate prior to concrete placement is contained in EC 75220 installation instructions, Sections D.3.3.2 and D.3.3.6. It should be noted that only very light surface rust has been observed on the exposed liner plate surface to date and does not appear to be of any concern from a structural integrity viewpoint.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/18/2010

Status: Closed

Date Closed: 8/27/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number:	114		
Individual Contacted:	Garry Miller	**Date Contacted:**	2/11/2010
Requestor/Inspector:	Anthony Masters	**Category:**	Question

Request: Are there any plans for monitoring or performing condition assessments from inside of the liner during or after de-tensioning or re-tensioning in regard to the identified bulges, etc? If so, what are those plans?

References:

Response Assigned to:	Paul Fagan	**Date Due to Inspector:**	

Response:

Deformation scans are to be performed on interior surfaces of accessible containment walls during the detensioning phase. The data obtained will be compared to the analysis results provided by PII. It is not the purpose of this monitoring to address the existing identified bulges. NCR 377992 is tracking the resolution to the identified bulges. (Ref. EC 75218, Section B.6.20 and Section E)

Misc Notes:

Response By:	Martin Souther		
Reviewed By:		**Date Response Provided:**	3/21/2010
Status:	Closed	**Date Closed:**	7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 115

Individual Contacted: Garry Miller **Date Contacted:** 2/11/2010

Requestor/Inspector: Anthony Masters **Category:** Question

Request: NCR 364655 does not appear to indicate which hole was being attempted to core. The adverse condition investigation form states that at the time of the report, it was impossible to determine if damage had occurred to the tendons. It further states this will be investigated during WO 1650505. Without noting the core hole boring attempt, how will the location or possibly affected tendon be identified? Also, at the time of this report, it stated this was the only occurrence. Were there any other occurrences related to this NCR?

References:

Response Assigned to: Paul Fagan **Date Due to Inspector:**

Response:

NCR 364655 has been updated to include the tendon designation for the affected tendon, 53H19. The location of the damaged tendon sleeve was documented in WOT 1636782-01 as the lower left pilot hole at core bore number 46. Tendon 53H19 is referred to in EC 75220 (Draft) and WO 1650505. There have been no other occurrences of penetrating a tendon sheath.

-----6/22/10 Update-------
Based on the elevation of the pilot hole for containment core# 46 it was assumed that the affected tendon was 53H19, but was not validated until the concrete was removed from this tendon which revealed the pilot hole.
EC 75220 has been issued which contained instructions for repairing tendon sheaths. In parallel to the EC, SGT has issued SGT NCR-1019 (with associated work instructions in WP3710A, WO 1690328-02) to document and all tendon sheath damage and the required repair plan for each damage location. A visual inspection by Progress Energy Civil Engineer of the tendon wires in the 53H19 repair area did not reveal any damage to the tendon wires. 53H19 tendon sheath was repaired under WO 1690328-02 and WO 1650505 was cancelled.
There have been no additional incidents associated with the Condition Assessment Core-Bore activities. NCR 364655 assignments have been completed as of 6/22/10.

Misc Notes:

Response By: Martin Souther

Reviewed By: **Date Response Provided:** 3/21/2010

Status: Closed **Date Closed:** 7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 116

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: 1)Refer to root cause failure mode 7.10, "Hydro-demolition Induced Cracking." Hydro-demolition (or other methods of concrete removal) of concrete structures (e.g. parking structure slabs, bridge decks, post-tensioned containment openings, etc) is typically performed in practice with the area being cut in a configuration that is subjected to only its natural body force (i.e. its dead weight) or in a state of minimal stress as the cut is initiated and progressed. In post-tensioned structures, the prestress in the area is brought to minimal levels (less than 5 to 10% of the original prestress), by controlled detensioning of an adequate scope of tendons in and around the opening area, prior to starting concrete removal. This was not the case when the concrete removal by hydrodemolition was initiated and progressed on the CR3 containment for creation of the SGR construction opening. The tendons were detensioned and removed only within opening area without any systematic sequence and, in fact, approximately 5 of the hoop tendons in the upper half of the opening area still remained tensioned when the concrete removal in the 8 ft x 6 ft mockup area at the bottom right corner of the opening was initiated. Therefore, SGR opening area at CR3 had a significant level of prestress in it and build-up of stress and energy gradients, with the most severe concentrations/gradients being at the corner areas, at the time the concrete removal was initiated at the bottom right corner of the opening. Failure mode 7.10 and related failure modes 7.8 and 7.9 do not consider and address the effect of destructive dynamic release and/or redistribution of stored energy from significant prestress and gradients in the SGR opening area at the time the concrete removal (by hydrodemolition in this case) was initiated and progressed. This effect would be most critical when the concrete removal is first initiated, especially in an area of steep stress (or strain energy) gradients. Please address this effect with regard to initiating and propagating the containment delamination at CR3.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The test area used on 9/30 was an 8' by 6' rectangle on the lower right hand corner of the opening. It was over 11 feet away from the closest tensioned tendon in the opening area and only 1.3 feet away from the nearest tensioned tendon below the opening. This test run did not challenge the concrete with delamination in the opening area.
On 10/1 the full hydrolaser activity began at 0430 for 75 minutes and then again at 0745 until 1415. However, the last tendon was de-tensioned at 1322. Thus if there were an issue it would have occurred prior to 1322. The test area demonstrated a removal rate of 48 cubic feet per hour. A photo confirms roughly that rate. Using this rate it is possible to estimate the concrete removal rate. At 1322 about 6 inches of concrete had been removed over the last tensioned tendon. The activity was secured at 1415 and a photo taken during that lull showed some of the horizontal tendon sleeves partially uncovered which would correspond to an average depth of about 5".
It is important to note that reviewing photos taken of the test area on 9/30 show cracks running from one horizontal tendon to the next. That is prior to the hydrolasing over a tensioned tendon so crack initiation was not caused by hydrolasing over a tensioned tendon. The photos taken just after the hydrolaser stopped at 1415 show chunks of concrete in all exposed areas. One would have expected damage to be centered on the highest part of the opening had there been a connection between hydrolasing and the last tensioned tendon.
Consider the condition of a tensioned tendon. It is held tightly by the interior concrete and is thus unlikely to move. While the degree of exposure varies in the afternoon photos, they indicate there is still solid contact between the horizontal sleeves.

The hour-glass shaped delamination region has two peaks. One is above the opening and the other is below it. Such a shape is unlikely if the delamination were precipitated or propagated by the hydrolasing of the last tensioned tendon at the top of the de-tensioned band for the SGR opening.
Overall the evidence indicates this did not play a part in the delamination.

Timeline located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #116

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 6/1/2010

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 117

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation S10-0004 (MPR Calc # 0102-0135-06), Revision 1, Section 4.4.3 "Thermal Loads". It appears that the calculation attempts not to use the maximum values of Pa and Ta for end-of-life and return-to-service design basis load combinations involving accident pressure and accident temperature for the containment concrete structure. Section 4.4.3 of the calculation references S&L calculation S06-0002 which makes a statement with no reference to the design basis document or the FSAR. Please provide clear reference and justification, consistent with the current licensing basis (CLB), for the values of pressure and temperature used in the design basis analysis.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Calculation S06-0002 restates the technical basis for the design requirements included in the FSAR without reference to the source. The response to S.I.T. Question 119 provides references to the FSAR bases to support the same approach. The design basis allows for consideration of the timing of peak conditions. The LOCA involves a normal operating gradient at the beginning of the event. Typically, peak pressure and peak liner temperature are combined with the normal operating thermal gradient on the concrete as a conservative case. As the liner temperature and containment pressure decrease from the peak values (after about 10 minutes), the temperature of the concrete begins to increase. After 10,000 seconds (~2.8 hours), the concrete temperature has peaked.

References: (see request folder for references)

Response to S.I.T. Question 119
Crystal River Unit No. 3 – PSAR Supplement 6, Analysis and Design of Tendons Anchorage Zone

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/13/2010

Status: Closed

Date Closed: 12/17/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 118

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: The design temperatures documented on page 14 on the CR-3 Containment Design Basis Document (DBD) has a 158 degree inside, 37 degrees outside for Winter Accident condition for the containment wall. The winter accident temperature profile for the CR-3 containment wall associated with the LOCA is described in FSAR Section 5.2.1.2.1 and the associated design load combinations are described in FSAR Section 5.2.3.2.1 and FSAR Table 5-3. The pressure-temperature blowdown analysis for the Reactor Building DBA is described in FSAR Section 14.2.2.5.9.

Calculation S10-0004 (MPR Calc # 0102-0135-06), Revision 1, Section 4.4.3 "Thermal Loads" uses the average winter normal operating temperature for design basis load combinations involving accident pressure (P) and accident temperature (Ta). This appears to be outside the CR-3 current design basis as described in the FSAR and the DBD. Please demonstrate and justify how the above treatment of the thermal load in the containment delamination design basis calculations, for load combinations involving accident pressure and accident temperature, is consistent with the current design basis as described in the FSAR.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The response to S.I.T. Question 119 provides references to the licensing bases to support a time history approach to the LOCA analysis. The LOCA involves a normal operating gradient at the beginning of the event. As the liner temperature and containment pressure decrease from the peak values (after about 10 minutes), the temperature of the concrete begins to increase. After 10,000 seconds (~2.8 hours), the concrete temperature has peaked. Treatment of thermal loads for the detensioning analysis (Calculation S10-0004) is described in the response to S.I.T. Question 101.

References:

Response to S.I.T. Question 119
Response to S.I.T. Question 101

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/13/2010

Status: Closed

Date Closed: 12/17/2010

Request Number: 119

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Issue

Request: Refer to Calculation S10-0004 (MPR Calc # 0102-0135-06), Revision 1, Section 4.4.3 “Thermal Loads”. It appears from the description in Section 4.4.3 (pages 12 & 13) of the calculation that only uniform (average winter operating) temperature is used in the design basis analysis and the temperature gradient across the concrete wall has been ignored with no justification provided. Ignoring the temperature gradient across the thickness of the wall is an incomplete treatment of the thermal load and not acceptable.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The design basis allows for consideration of the timing of peak conditions. The LOCA involves a normal operating gradient at the time of peak pressure and peak liner temperature. As the liner temperature and containment pressure decrease from the peak values (after about 10 minutes), the temperature of the concrete begins to increase. After 10,000 seconds (~2.8 hours), the concrete temperature has peaked and the impact of the gradient to concrete tensile stress is only about 13% (1) above the normal operating condition. (1) Note: the 13% value was changed from 12% on 10-14-2010 due to final analysis contained in S10-0062, “Operating and Accident Thermal Gradient for Concrete”
(not approved).

(Supplemental paragraph 11/29/10): A consequence of a time-history approach to LOCA analysis is the need to consider individual components of the accident (P alone or T alone). Calculation S10-0062 also summarizes the effect of separate accident pressure and temperature
conditions.

A description of the thermal analysis performed associated with detensioning and repair of containment is provided below with reference to design basis documents.

Detensioned State:
The condition that exists while the repairs are being made (detensioned state) only includes a mild gradient. S10 0004, Tendon Detensioning Calculation, (MPR Calc #0102-0135-06) uses 10 degrees. EC 75218 implements controls to ensure that unprotected, unreinforced concrete exposed by removal of the delaminated concrete is not exposed to thermal conditions that would cause a 10 degree gradient.

Preliminary Evaluation of Long Term Effect of Detensioning:
Consistent with original design, an operating thermal gradient was used in S10-0004, Tendon Detensioning Calculation, to evaluate the potential end of life effect. This is discussed in Reference 1 (FSAR Chapter 5, Reference 19):

The design pressure inside the Reactor Building during LOCA is equal to 55 psig and the maximum pressure was initially calculated to occur at approximately 200 seconds after initiation of the accident. The maximum liner temperature of 280 F was initially calculated to occur at approximately 650 seconds after initiation of the accident. However, the maximum temperature gradient which causes the most severe thermal stresses, was calculated to occur at 10,000 seconds after initiation of the accident.

Consequently, the determination of the most severe stress state, due to thermal and pressure loads, requires that a number of stress analyses be conducted at various elapsed times after commencement of the accident.

Reference 1, Section 4.4 also describes a conservative approach where the maximum values for accident temperature and pressure were superimposed to avoid the need to alter the stress analysis due to timing changes that were expected as further analyses were performed:

A conservative estimate of the stresses was obtained, by superimposing directly the maximum values for accident temperature and pressure. Consequently, although subsequent calculations alter the time at which minimum [sic] pressure and liner temperature occur, these changes do not affect this analysis.

The conservative analysis was performed for convenience and is not design basis.

For the forward looking analysis performed in Calculation S10-0004, the limiting condition chosen for analysis is the case that includes peak pressure and peak liner temperature. At that point in the accident scenario, the concrete thermal gradient is adequately represented by normal operating conditions. This selection is also justified by further analysis to assess the impact of the difference between normal operating thermal gradient and the accident thermal gradient for 10,000 seconds. The profile for accident thermal conditions from Reference 1 Figure 14 is included as Attachment 1.

The figure shows the mid-surface and external surface temperature to be unaffected during 10,000 seconds. Only the inside 14 inch thickness is affected. For the purpose of determining required reinforcement, tensile stresses are important. The thermal gradient creates tension on the outer half of the section. The average temperature for the accident gradient is only a few degrees higher than the average temperature for normal operating conditions. The □T (exterior temperature to average temperature) for accident conditions is 13%(1) higher than for normal operating conditions. Thus the stress increase is 13%.(1)

Reduction in pressure more than compensates for the effect of □T increase over the same period.

This approach is also consistent with the method used for GAI Report No. 1913 “Reactor Building Dome Delamination Report” December 10, 1976 (reference Section 4.4 and Table 2-3).

Final Design:
Design of reinforcing steel for Bay 34 uses ACI 505-54 for the effects of thermal gradient. Consistent with original design, those effects were added to the computer analysis results. Conservatism was included by use of the accident thermal gradient for 10,000 seconds.

REFERENCES: (see request folder for references)

1. FSAR Chapter 5, Reference 19, “Analysis and Design of Tendon Anchorage Zones”, Gilbert Associates, Inc., Report No. 1730, 1970, (Crystal River Unit 3 PSAR Supplement 6, Volume 5).
2. GAI Report No. 1913 “Reactor Building Dome Delamination Report” December 10, 1976.
3. S10-0062 “Operating and Accident Thermal Gradient for Concrete”.

ATTACHMENT:
1. Reference 1 Figure 14, in part.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/13/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Status: Closed

Date Closed: 1/10/2011

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 120

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: At the end of detensioning and retensioning operation, the stresses in the containment wall due to dead load and prestressing force redistribution are locked in. It is not clear from Calculation S10-0004, Revision 1, how the end-of-life condition was evaluated using the minimum required prestressing loads. Please explain.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The model was loaded incrementally to account for the cumulative effect of sequential load steps as described in Section 4.5.1. Stresses resulting from the application of one load step are retained in the model as the starting condition for the subsequent load step. Therefore, all locked in stresses are accounted for in the end of life condition of the building. After tendons are retensioned, , tendon forces are reduced to end of life values (Table 4.5) and accident loads are applied. The results are included in Tables 8-13, 8-14, 8-18, 8-19. Several end of life conditions are evaluated further in Table 8-15. It should be noted that the 60 year end of life forces listed in Table 4.5 have been revised in subsequent calculations.

As discussed in the purpose statement (Section 1.2), Calculation S10-0004, Tendon Detensioning Calculation, is intended primarily to evaluate the building in the de-tensioned condition. For the end of life condition, S10-0004 is superseded by later calculations:

References:
S10-0004 Tendon Detensioning Calculation
S10-0012 Stresses around the SGR Opening due to Design Basis Load Cases (see request folder for reference)
S10-0015 Equipment Hatch Sub-model (Refined Model) (see request folder for reference)
S10-0030 Reinforcement Design for Delaminated Containment Wall (see request folder for reference)
S10-0057 SGR Construction Impact of Containment Stress (calculation is not approved)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/17/2010

Status: Closed

Date Closed: 10/19/2010

Request Number: 121

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to Calculation S10-0004, Revision 1, the regression analysis plots in Attachment 1C for hoop tendons uses data from only 6 of the 8 surveillances conducted so far. Data from the first two surveillances were not used for hoop tendons, whereas these were used for the vertical and dome tendons.

(a) Please explain and justify the selective use of surveillance data in the regression analysis of hoop tendons.

(b) Provide and reference the technical source document for the regression analysis of the hoop, vertical and dome tendons.

(c) Indicate the tendon force values from the regression analysis that were used in the design basis analysis for each tendon type.

(d) The X-axis of the regression analysis plots is based on "Ln(T, Years since SIT)" and not "Ln(T, Years since initial tensioning)," which is the appropriate time scale for developing a trend for prestress forces. Please explain. Also, in Table 4.5, please clarify the time from date of initial tensioning that correspond to the two cases under the "Time" column.

Follow-up question:

Reference the follow-up question for S.I.T. Question 89. Ensure that consideration is being given to the age of tendons when performing regression analysis. Use of the SIT date may change predictions.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

(a) The trends developed for S10-0004, Tendon Detensioning Calculation, are those documented in Calculation S10-0026, Tendon Regression Analysis for Detensioning Only, and based on a data summary included in the vendor report on the 30 year tendon surveillance. These trended forces were used to support de-tensioning calculations and do not represent final design. S10-0028, RB Tendons/Forecast End of Life Force, developed updated trends using data documented in all eight of the individual 1, 3, 5, 10, 15, 20, 25 and 30 year surveillance reports and screened to eliminate invalid values (e.g. lift-off forces documented for tendons that were de-tensioned and re-tensioned during prior surveillances) These updated trends are shown in Attachment 2 to Calculation S10-0028. Attachment 2 includes tabulated lift-off data as well as plots that show individual data points, trend lines and trend line equations.

The group mean forces computed in S10-0026 for the time (T = 33.3 years since the Reactor Building SIT) of de-tensioning are close to those determined for the same point in time using the S10-0028 trends that were developed using all valid surveillance data. Computed forces are listed below for comparison.

Group	Mean Force per S10-0004 / -0026	Mean Force per S10-0028
Vertical	1,529.6 kip	1,510.0 kip
Hoop	1,365.8	1,381.1
Dome	1,380.1	1,375.3

Because these forces are similar for the two calculations, it is acceptable to use the forces developed in the regression analysis for detensioning calculations.

(b) As discussed in (a) above, the group mean tendon forces shown in S10-0004 are those computed in S10-0026 using the lift-off data summary included in the 30 year surveillance report. The regression computations were performed by Microsoft EXCEL. EXCEL computes trend line coefficients using the method of least squares as developed in mathematical and engineering statistics texts (e.g., Probability and Statistics for Engineers by Irwin Miller and John Freund; Prentice-Hall, 1965).

(c) Tendon force values used in the de-tensioning analyses are expected means shown in S10-0004 and computed using the trend line expressions developed in Calculation S10-0026. Design basis calculations, which were done subsequent to the issue of S10-0004, forecast group mean tendon forces using the expressions shown in S10-0028. These expressions were derived using lift-off data documented in the reports covering the individual surveillances and screened for validity as discussed in (a) above. Design minimum required prestress values are used in design basis analysis except where the load conditions require forecast values (e.g., test conditions).

(d) Since tendons are tensioned over an extended period, a plot with a 'Time Since Tensioning' abscissa has an origin that is not fixed at a given point in time. For this reason, trend plots always use years since the SIT (a fixed date that determines surveillance performance windows) as the time origin basis. In S10-0004 Table 4.5, 'Return to Service After SGR' is an estimated post-repair start-up date in mid 2010, approximately 33 ½ years after the November 1976 SIT. The '60 Year End of Life' date is in December 2036.

Forecast end of life (based on the December 2036 expiration of the pending extended operating license) group mean tendon forces were evaluated to verify that these exceed design minimum required pre-stressing levels.

Attached: (see request folder)
S10-0026, Tendon Regression Analysis for Detensioning Only
S10-0028, RG Tendons/Forecast End of Life Force
S10-0028, Attachment 2, Tendon Mean Force Trend Development

Follow-up response (11/3/10):

A study was performed to assess the impact of using the pre-operational SIT date as the zero time origin. The result was a 1.2% change in the tendon force predicted at the end of extended license (60 years). Use of the surveillance date (one day prior) as the zero time origin for trending would have a similar effect in the opposite direction than described above. Therefore forecasts associated with regression analysis will continue to use the pre-operational SIT date as the zero time origin. This conclusion is specific to CR3 tendon tensioning data.

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	9/27/2010
Status:	Closed	**Date Closed:**	11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 122

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation S10-0004, Revision 1. The calculations are not documented in a fully traceable and transparent manner and seems to be deficient in quality standards and design control standards for reasons such as the following.
(a)Emails are being referenced as design input sources, especially for numerical design inputs, rather than technical source documents. Design inputs such as regression analysis of tendon surveillance are included with no verification and no reference to the source for the data. Design inputs should be referenced in calculations to the technical source documents.
(b)Calculations are based on computer models and analyses, and report and discuss results from these computer analyses. However, these computer models and analysis inputs, outputs, extracted results, etc. are not included with the calculation, even in electronic form.
(c)Draft documents are referenced as design input source in a final approved calculation (e.g. Reference 10 in the calc S10-004, Ref. 9 in MPR calc. 0102-0135-04)
(d)Units used in the calculation not clearly identified, especially in the figures, and global co-ordinate system for the model not identified.
Please address quality and design control standards in design calculations consistent with the 10 CFR 50 Appendix B program. Please address this concern, as applicable, in all design basis calculations related to the CR3 containment delamination issue.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Quality requirements are implemented consistent with importance to safety and approved procedures. Some early calculations that support concrete removal and detensioning are brief with regard to documentation but no less rigorous regarding analytical tools and technical basis brought to bear for decisions implemented. Because Calculation S10-0004 supports detensioning tendons to allow for repair of the structure and evaluates interim conditions until the final repair and final design basis calculations are approved, documentation on some items may be brief.

However, all calculations are prepared in accordance with approved procedures as required by Progress Energy Quality Assurance Program and the Quality Assurance Programs of our consultants as accepted by Progress Energy. Progress Energy Nuclear Generation Group Procedure EGR-NGGC-0003 requires an Owner Acceptance Review. During this review process, the sources for design inputs are validated and the reviewer provides his/her signature to confirm, among other requirements, that the design meets the requirements established by the design inputs. The results of the owner reviews are documented in Progress Energy approved calculations.

Analyses are documented per ANSI N45.2.11-74 and approved procedures. Where the calculation record is abbreviated for clarity or because computer output files are too lengthy, references to additional computer input/output files have been provided per MPR's QA program.

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	10/28/2010
Status:	Closed	**Date Closed:**	10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 123

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Refer to Calculation S10-0004, Revision 1. The second paragraph on page 16 makes a statement that: "These (interaction) diagrams do not incorporate a safety factor into the force and moment limits." Please confirm whether or not the interaction diagrams include the material capacity reduction factors (Φ) applicable in ultimate strength design. If not, please provide justification.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The interaction diagrams referenced in calculation S10-0004 were developed in calculation S10-0005, Interaction Diagrams for Selected Sections. Per Page 12 of calculation S10-0005 the capacity reduction factor φ was calculated based on J. Wight and J. MacGregor, Reinforced Concrete Mechanics and Design, Pearson Education, Inc., 5th Edition. The methodology used in this reference is based on ACI318-05, Section 9.3. The capacity reduction factors in ACI 318-2005, 9.3.2 are equivalent to or more conservative than in ACI 318-63, 1504.

Reference:
S10-0005 R1: Bending/Tension Interaction Diagrams for Selected Sections

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/23/2010

Status: Closed

Date Closed: 10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 124

Individual Contacted:

Date Contacted: 3/25/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: The description in Calculation S10-0004, Revision 1 (Section 7.1.3, 4.5.1 etc) and in MPR Calculation 0102-035-04, Revision 0, although not clear, seems to imply that the liner is included as a structural element for the CR3 containment end-of life and return-to-service evaluations. If true, this would be outside the current licensing basis (CLB) for CR-3. Please explicitly explain and clarify the treatment of the liner under design basis loads and load combinations for CR-3 containment delamination design basis calculations and how it is compliant with the CLB.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The content of this question is included in the content of question #67. Refer to Question #67 for the detailed response to this Question. This question may be closed.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/28/2010

Status: Closed

Date Closed: 10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 125

Individual Contacted: Charles Williams

Date Contacted: 4/9/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With regard to the evidence sheets for failure mode FM 6.5, the NRC SIT has the following comments with respect to certain inaccurate statement of facts.

a.In the Description Section reference has been inappropriately made to RG 1.35.1 for acceptance criteria for tendon lift-off measurements during surveillance. The correct reference is ASME Section XI, IWL-3221 and the calculation that developed the predicted forces for the IWL-3221 criteria. RG 1.35.1 only provides a methodology or procedure for calculating tendon losses in arriving at predicted forces for use in inspection programs, but the material properties that go into these calculations are plant-specific. Also, the methodology used by CR3 is a little different from the methodology in RG 1.35.1. The actual criteria are in the codes and the plant-specific calculations that develop the code-based acceptance by examination criteria. Also, RG 1.35.1 has been stated as being requirements. Note that regulatory guidance are not requirements unless included by reference in plant Technical Specifications. Otherwise, licensees may voluntarily adopt guidance in RGs.

b.Item b. under “Verified Refuting Evidence” section makes a statement: “The regression analysis performed by PSC and PE on measured tendon forces show a better correlation than the predicted curves (FM 6.5 Exhibit 9).” The “Discussion” section also makes a similar statement that “A regression analysis was used instead to approximate tendon force losses that match better with measured lift-off forces.” These statements are technically misleading because you should not be comparing a best-fit curve (regression) using measured lift-off data to the same lift-off data to draw a conclusion that the curve so arrived provides a better match. Please rephrase these statements to make technical sense.

c. Item b. under “Verified Refuting Evidence” section makes a statement: “It also indicates that RG 1.35.1 used with the CR3 DBD numbers may have underestimated the expected force loss, particularly for the horizontal tendons, resulting in overly…………..” This statement should be making reference to the specific CR3 calculation that developed the predicted tendon force curves for use in tendon surveillance and not to RG 1.35.1. The issue with regard to predicting tendon forces at CR3 is not with the regulatory guidance but with the plant-specific calculation that developed the predictions. In fact, the CR-3 calculation that predicted tendon forces claims that the methodology used therein is superior to that in RG 1.35.1.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Comments have been incorporated into the revised FM 6.5.

Misc Notes:

Response By:	Charles Williams		
Reviewed By:		**Date Response Provided:**	6/3/2010
Status:	Closed	**Date Closed:**	6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 126

Individual Contacted: Don Dyksterhouse

Date Contacted: 5/6/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request:

Calculation S10-0019 documents a current design compressive strength of 6700 psi for the original containment concrete based on statistical evaluation of the 90-day cylinder test results and crediting age strengthening (by 15%) between 90 days and the present time based on limited core bore tests.

1.Please provide information on what this increased strength will be used for in the design basis analysis for the repair of the containment delamination at CR-3.
2.If the increased compressive strength is being used to increase the tensile capacity of concrete in design, justify the use of an increased tensile capacity of concrete for the delamination repair given that CR-3 has had an operating experience of two containment delaminations (dome and cylinder) where weakness in tensile strength of the containment concrete were a significant contributing factor.
3.Justify the age-hardening credit factor of 15% that is: (a) based on limited core bore tests that are neither sufficient nor representative of the entire containment structure to achieve a 99% probability of the specified strength being exceeded; (b) based on methods in ACI codes meant for evaluating and qualifying deficient concrete at the time of original construction, and (c) minimizes margin and conservatisms contrary to the defense-in-depth regulatory philosophy for design of nuclear safety-related SSCs.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Calculation S10-0019, Design Compressive Strength for RB External Cylinder Walls and Buttresses, has been revised (Revision 1) to reflect an increase in current concrete compressive strength from 5000 psi to 5800 psi based on a statistical evaluation of the 90-day cylinder test results. No credit has been taken for age strengthening as was previously proposed.

1. The increased compressive strength of f'c =5800 psi has been used in the reinforcement design (Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall) for calculating the shear and compressive strength capacity of the containment wall and developing the associated strength interaction diagrams. The increase compressive strength was also used in Calculation S10-0021, Rev. 6, “Concrete Radial Reinforcement” when evaluating the radial anchors for group effects, i.e. minimum anchor spacing less than critical spacing. The interaction diagrams are based on cracked section analysis per ACI 318, Section 1503 where the strain is assumed zero at the section’s neutral axis and varies linearly throughout the depth of the member. The tensile strength of concrete is neglected and the usable compressive strain at the extreme fiber is limited to 0.003 in/in. The increased compressive strength has not been used to increase the allowable tensile strength of the concrete (Refer to Response #2 below).

Note that the evaluation of increased concrete compressive strength was similar to the process used to evaluate the in place concrete compressive strength for the CR-3 containment dome delamination event in 1976. At that time, the process of establishing a higher minimum compressive strength of 6000 psi was demonstrated and accepted by Supplement 2 to the original CR-3 Safety Evaluation Report. Therefore, that process is part of the CR-3 Licensing Basis.

2. In the repair area, the increased compressive strength of the concrete has not been used to increase the allowable tensile strength of the concrete. In fact, the tensile strength of the concrete has been ignored in the repair area.

3. S10-0019, Revision 0, included the age-hardening credit factor of 15%. However, Revision 1 of this calculation eliminated this factor.

NOTE: The 50.59 Evaluation for Containment Repair will address the (additional) question of whether increasing the compressive strength from 5000 to 5800 psi results in changing or exceeding a design basis limit for a fission product barrier.

(see response folder for Calculation S10-0019 and S10-0030)

Misc Notes:

Response By: Don Dykstérhouse

Reviewed By:

Date Response Provided: 8/10/2010

Status: Closed

Date Closed: 8/27/2010

Request Number: 127

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to Root Cause Failure Mode 1.1 “Excessive Vertical and Hoop stresses / High Prestress of 1.6”, the NRC SIT has the following comments.

(a)The comparison of prestress for different plants made in Exhibit 5 is based on prestress forces for different plants tabulated in Exhibit 4 (Table 2-1 of BC-TOP-5). It is to be noted that the effective prestress values in Table 2-1 of BC-TOP-5 are “minimum design” effective prestress values, which may be an arbitrarily chosen value for a plant after accounting for all prestress losses and allowing a margin. However, the hoop tendon force of 1325 k in Exhibit 5 used to calculate the effective prestress value of 793 k/ft is not the minimum design value. So the comparison being made in the exhibit is between apples and oranges. The minimum design force for hoop tendons for CR3 is 1252 k (See CR3 DBD). If the minimum design force of 1252 k is used, then the minimum design effective prestress would be 1252 k x 94/157’ = 750 k/ft (instead of 793 k/ft based on average spacing of tendons (= cylinder height/# of hoop tendons along height). If the typical spacing of tendons of 19.5” or 1.625 ft is used (based on two tendons every 39”), this force would be 1252 k/1.625’ = 770 k/ft. The hoop force corresponding to the CR3 design accident pressure of 55 psig would be 55 psi x (144 in2/ft2) x 65 ft radius / (1000 lb/k) = 515 k/ft. Therefore, the minimum design prestress ratio for CR3 would be 770 / 515 = 1.5 or 750 / 515 = 1.46, and not the higher values of 1.6 or 1.54 reported in the FM. The minimum design prestress ratio of 1.5 at CR3 is not unique to CR3 since there are other containments in the industry that have been designed to this ratio (e.g. ANO). Since the tendons in all containments are originally locked off to a stress level of approximately 0.7Fu, the actual prestress and the prestress ratios in the containment concrete at any given time is expected to be higher than the minimum design values and are comparable for many post-tensioned containments. What may be different about CR3 containment is that it seems to have lesser amount of reinforcement compared to other containments designed for a prestress (minimum design) to design accident pressure ratio of 1.5.

(b)The prestress pressure to design accident pressure ratio is not a true or correct measure of the level of prestress in containment concrete for comparison since the actual concrete stress level from prestressing tendons is a function of the wall thickness, the spacing and location of the tendons, the force in the tendons (depends on # wires, lock-off stress, losses etc) and other factors. Since design accident pressures can be different, one can have two containments with the same prestress to design accident pressure ratio but different prestress levels in the concrete. The potential for prestress level in the concrete to cause delamination is dependent on the actual stress in concrete wall due to prestressing tendon forces, and not the general prestress to design accident pressure ratio. Therefore, to establish a conclusion that the prestress level was relatively higher at CR3 in comparison to other similar containments, the comparison should ideally be based on actual stress due to prestressing tendons in the containment concrete at given time (e.g., at initial tensioning, at SGR, end of life) of comparison. The radial tension and inward prestress pressure due to hoop tendons are a function of the radius at which the hoop tendons are located in the wall thickness.

(c)Although the title of the FM includes excessive vertical and hoop stresses, neither the body of the FM nor the conclusion addresses the potential role of the vertical prestress level in the delamination.

(d)The end-of-life vertical tendon force of 1325 kips used in Exhibit 8 does not seem correct. Regression analysis of surveillance date forecasts it to be of the order of 1484 k for vertical tendons.

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

You are correct. Section b of the calculation should have been labeled "Comparison with the Original Gilbert Cilbert assume the tendon was at 60% GUTS and only 95% of the tension be available to the wall in an accident. That is the source of 1325 kips.

Using the Minimum Design Force of 1252 kips would have been consistent with the Bechtel table but, as you point out, the Minimum Design Force is hardly a uniform standard.

A better comparison would be to use 70% GUTS for each containment. As we realized, however, the comparison would still be simplistic since hoop stress is not directly tied to delamination. We recently chose to use the same technique that was used in the dome delamination event. It is Compressive-Tensile Interaction. It indicates a small impact. The real issue is the peak stresses which falls both in FM 1.1 and FM 1.15.

c) Near the bottom of the discussion section, we comment, "Forces and stresses generated by the vertical tendons are only half those generated by the hoop tendons."
They were considered and determined to not be significant.

d) We agree. The title used for that section was misleading. The calculation has been corrected. Section C addresses end-of-life.

We would enjoy the opportunity to discuss this with you in person.

Misc Notes:

Response By: Charles Williams

Reviewed By: **Date Response Provided:** 5/27/2010

Status: Closed **Date Closed:** 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

Request Number: 128

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to failure mode FM 1.2 “Excessive Radial Stresses/No Radial Reinforcements,” does not fully address the role of vertical prestress in producing radial tension in the plane of the hoop tendons. Although the use of the calculation (S10-0003) in Exhibit 6 in its current form is questionable, the results for the “vertical only” case seems to contradict the conclusion in Item 1 of the “Discussion” section of the FM that the vertical compression does not generate radial tensile stresses. Please address: (a) the role of the vertical prestress in producing radial tension at the hoop tendons definitively; (b) the role of biaxial compression – tension interaction phenomenon in producing the delamination in the CR containment wall.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The role of stress in all three dimensions is incorporated in the Abaqus computer code and the prediction of cracking and delamination stem from that code. The FM concludes that radial tensile stresses did contribute to delamination. We do not see delamination cracking between vertical tendons the way we see cracking between horizontal tendons. This is probably due to the wider separation between vertical tendons, the smaller concrete displacement effect, and smaller curvature in the azimuthal direction compared to the curvature in the vertical direction.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 7/21/2010

Request Number: 129

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With Reference to root cause failure modes FM 2.12, “Inadequate Strength Properties” and FM 3.4 “Inadequate Aggregate,” an important piece of physical evidence that was observed but not discussed in the evidence sheets is that: Physical examination of the CR3 delaminated surface for both the 1976 dome delamination and 2009 cylinder wall delamination show that the delaminated surface (or fracture plane) traversed through the coarse aggregate and not around the aggregate, as would be expected in normal concrete. This indicates that the coarse aggregate was the weak link in the initiation and/or propagation of the delamination crack. In typical concrete used in the nuclear industry, the cement mortar is the weak link to cracking and the coarse aggregate helps arrest cracks and therefore any crack will have to propagate around the coarse aggregate. Thus, for these concretes the surface area of the fracture plane resisting cracking or crack propagation is higher and therefore the concrete exhibits higher tensile strength properties that provides better resistance to cracking and crack propagation. For the CR3 concrete the coarse aggregate, being the weak link against cracking, could have enabled a more significant contribution to the initiation and propagation of the delamination. Please address the implication of the above stated observed evidence with regard to tensile strength properties (FM 2.12) of the CR3 concrete and the role of the soft aggregate (FM 3.4) in producing concrete with relatively lower tensile strength and reduced crack arresting capability at CR3.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

We certainly agree that the observation of cracks traversing across the aggregate is important. The FM discusses the issue in two places. These sections are copied below:

8. Analysis of concrete strength records lead to the conclusion that aggregate strength limited concrete strength development. Grout cores (Concrete without the coarse aggregate that used to start each pour) tested significantly stronger than concrete from the same pour (FM 3.4 Exhibit 10 page 1-2 shows selected concrete test results while page 3 shows the test results for grout cores) .
These observations indicate that the presence of aggregate limits the tested strength of the concrete. This conclusion is further validated based on the observation of core fracture – where all cracks propagate through the aggregate and not around the interface zone.
Although the concrete at CR3 is in full compliance with design criteria (5000 psi at 28 days) the aggregate limited the concrete’s ability to reach the full strength potential of its low W/C and cement paste properties.
9. The relationship between compressive strength (fc) and modulus of elasticity (Ec) has many different expressions in the literature and codes. Using these empirical relationships, the CR3 concrete would be expected to have Ec in the range of 4.75 to 5.25x106 psi. However, the measured Ec was substantially lower, averaging 3.4x106 psi. This can be explained by a lower than normal modulus of elasticity for the aggregate.
In general, aggregate properties will affect concrete Ec much more than its compressive strength. Accordingly, concrete made with weaker aggregate can meet strength specification while failing to achieve the Ec calculated from design formulas.
10. The aggregate impact on creep/shrinkage properties is governed by two mechanisms – it will either restrain creep by providing a volume of inert hard material, or increase creep by providing weakened interface zone on its surface. For the same aggregate volume, fine aggregate will have more surface area than coarse aggregate. Therefore, it will require more cement paste with larger effect on the creep properties. At CR3, the aggregates’

grading resulted in increase of the finer fractions with the corresponding increase of creep and shrinkage potential. The presence of softer coarse aggregate reduced its ability to restrain that creep and shrinkage.
11. Normally, the coarse aggregate serves as a crack-arrestor in the concrete thanks to its higher strength and density. The presence of porous and fossiliferous particles introduces weakened locations where cracks may originate, or where existing cracks may propagate. The CR3 concrete had up to 50% of these weaker particles, which would have significant influence on its ability to stop propagation of cracks.
This would have a detrimental effect on the tensile strength and fracture energy of the concrete, especially when subjected to high tensile or shear stresses.
12. Analysis performed by PII in 2010 determined that reduced tensile strength properties of the concrete contributed to the delamination.

Note that PII found this failure mode to be confirmed.

FM 3.4 discusses the impact on direct tensile capacity due to aggregates and PII determined that this Failure Mode did contribute to the delamination.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 130

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to FM 2.12 “Inadequate Strength Properties,” how does the concrete material properties and specifically strength properties including the tensile strength (direct and split) of the CR3 containment concrete compare with those of post-tensioned containments that have successfully executed creation and restoration of SGR construction openings or with those of similar containments?

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

What we found is that CR3 concrete meets design expectations for tensile strength. We did find that other aggregate types provided better tensile capacity and concluded the aggregate at CR3 contributed to the delamination.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 131

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to root cause failure mode FM 4.5, “Excessive Creep,” the information presented for the failure mode indicates that the portion of the CR3 containment wall concrete inside the prestress. However, at the same time the wall layer outside the plane of the hoop tendons would experience creep under sustained radial tension due to hoop tendons and Poisson’s effect from the vertical and hoop prestress (biaxial compression – tension interaction), which could be more severe or comparable to the creep in compression and could have an effect of pulling apart at the plane of the hoop tendons and contributed to the delamination or in causing degraded concrete properties at the hoop-tendon plane. It is reported in literature that concrete deforms more in tension than in compression and that, for a given applied stress level and/or applied stress to strength ratio, tensile creep is higher than compressive creep. The mechanism of tensile creep is related to the opening of microcracks, which may explain the higher tensile creep of concrete. Under tensile stress the weakest part of the concrete (i.e. the flaw(s) or defect(s), such as microcracks) will propagate and may have contributed to the degraded mechanical properties of the CR3 concrete. Please address the potential effect of creep in tension, if any, in contributing to the delamination such as facilitating degraded concrete properties (such as larger concentration of micro-cracks) at the plane of the hoop tendons where the delamination occurred.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Tensile stress can be very high at the tendon sleeve holes but only over a very limited range. In terms of bulk movement of concrete the tensile stress is low enough to cause creep to not be a contributor to the delamination. Creep is specifically modeled in the computer model so any effect that does occur is included in the simulation.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 132

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to FM 7.1, clarify if the phenolphthalein tests and SEM observations made on the specimen(s) taken from areas including the hoop tendons sleeves show indications of a pre-existing crack at both the 6 o'clock and 12 o'clock locations, or was only one of the locations investigated? Clarify whether the micro-crack indicated in page 5 of FM 7.1 Exhibit 1 was at the 6 o'clock or 12 o'clock location of the specimen from the tendon sleeve interface.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The concrete samples were collected during the SGR concrete removal activity. We could roughly identify that it was next to a horizontal tendon but we cannot place it exactly. It could not be determined whether the crack location of this specimen was from the 6 or 12 o'clock position.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 133

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to failure mode FM 7.1 "Damaged Properties / Tensile Stresses from SGR" and FM 1.15 "Inadequate Design Analysis Methods for Local Stress Concentration Factors (SCF)," address the contributions of the prestress forces in the vertical tendons and hoop tendons at original tensioning in producing tensile stress concentrations that potentially resulted in the pre-existing cracks that seem to be observed in the carbonation study at the 6 'o clock and 12 0'clock locations around the hoop tendon sleeves? What were the maximum tensile stresses produced at these locations around the tendon sleeves by prestress in (a) the vertical tendons and (b) the hoop tendons? What were the maximum compressive stresses at the 3 o'clock and 9 o'clock locations resulting from vertical and hoop prestress and possible ovaling of the tendon conduit (Note that the tendon conduits are likely exhibit flexible pipe behavior since D/t ratios are 84 and 42, respectively, for thicknesses of 1/16" and 1/8", and the bond between smooth conduit surface and concrete is weak – this could have contributed to pre-existing cracks at the tendon sleeves. One would expect the tendon sleeves to help prevent cracking due to stress concentrations around the sleeves if the bond at the sleeve and concrete interface is strong).

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

This is a case where a picture helps a lot. The area around a representative horizontal tendon is shown in Figure 3.4. This is a critical area due to the high stresses and displacement of concrete by the tendon sleeves. This is the location along which the delamination propagated from the top and bottom of the holes to the next hole vertically or propagated circumferentially (azimuthally) around a segment of the building.
Figure 3.4 Stress contours for normally tensioned conditions at the intersection of a vertical and horizontal tendon.
horizontal tendon. Figure 3.5 Stress contours for normally tensioned conditions in between intersections of a horizontal tendon with vertical tendons

These figures have been copied from the most recent revision of the root cause report. Figure 3.4 is situated at the high stress interface of a horizontal and a vertical tendon. Figure 3.5 is at the lower stress location in between vertical tendons but you notice the high stress that still occurs at the horizontal sleeve hole.

Our experience is the bond between the sleeves and concrete are weak as evidenced by the clean separation of the sleeve from the concrete during concrete removal.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 7/21/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 134

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Failure modes FM 1.2, FM 1.15, FM 7.1, FM 7.5, and possibly others use or refer Calculation S10-0003, Rev 0 (MPR Calculation No. 0102-0135-05) "Containment Repair Project – Conduit Local Stress Analysis" (see FM 1.2 Exhibit 6, FM 1.15 Exhibit 6, FM 7.5 Exhibit 2) as evidence in support of the root cause investigation/determination. During preliminary NRC calculation review and discussions with PE design basis analysis team, it was indicated that this calculation will not be used for any purpose. The NRC SIT finds that the use of the calculation in its current form in the root cause investigation is questionable. In addition to the issue raised in NRC SIT Request #96, the axisymmetric finite element model and the results/conclusions of this calculation seem to be questionable for the following reasons:

(a)The calculation has a very limited purpose (i.e., to determine if the absence of either the vertical and horizontal (prestress) compressive load results in a more limiting stress condition around the conduits) and concludes that it does not provide a basis for the detensioning sequence. The results indicate that the initial configuration with all vertical and hoop tendons tensioned provides the maximum tensile stresses around the hoop tendon conduits and, therefore, the detensioning sequence does not matter which is contrary to the fact that CR3 did have to go through with an elaborate detensioning scheme and sequence to ensure that the delamination did not propagate. The root cause investigation is using the numerical results/conclusions of the calculation for a purpose other than the stated purpose without authenticating the correctness of the calculation and the results. The calculation results indicates that the prestress from the vertical tendons produce the maximum tensile stresses (potentially radial) around the hoop conduits, which are significantly higher than that produced by the hoop tendons, and may have had a very significant effect in causing the delamination. This needs to be definitively investigated (please address). The FM 1.15 discussion seems to indicate that this kind of radial stresses could be generically present in all post-tensioned containments. However, a delamination was observed only at CR3, among about 10 to

(b)The axisymmetric model based on a 39-inch vertical segment of the cylindrical wall is not a true or complete representation of the physical stiffness of the wall at the location of the SGR construction opening, where the delamination was observed. The use of a 39-inch vertical segment and the chosen boundary conditions seem to grossly distort (exaggerate) the axial and flexural stiffness of the containment cylindrical wall, and specifically more so in the vertical direction. The vertical prestress load causes bending in the cylindrical wall due to its eccentric location with respect to the centerline of the wall thickness. Further, detensioning of both hoop and vertical tendons cause bending (bulging) in the wall. The axisymmetric model used may at best be representative of the bottom segment of the wall near the base mat, where radial reinforcement was provided in the original design.

(c)The "Horizontal only" case in the axisymmetric model means that all the 144 vertical tendons in the containment are detensioned, while all the horizontal tendons remain tensioned 360 degrees along the perimeter and all along the height. Only a limited number of vertical tendons were actually detensioned.

(d)The "Vertical only" case in the axisymmetric model means that the hoop tendons are detensioned 360 degrees at the same time, while all the 144 vertical tendons are in the tensioned state, which is also a deviation from actual detensioning.

(e)The calculation does not account for the dead load on the wall, which is a realistic load that always exists with prestress loading.

(f)The vertical and horizontal tendon forces of 1474 k and 1398 k, respectively, at the end of the SGR outage (approx. 33 years from SIT) are not representative of the actual tendon surveillance data. The regression analysis of CR3 tendon surveillance data documented in Calculation S10-0026 determined the mean vertical and horizontal tendon forces at 33.3 years since SIT to be 1530 k and 1366 k, respectively. Also, the impact of higher hoop and vertical tendons forces at the time of lock-off has not been evaluated. The calculation results indicate that, under prestress loads, much of the section is in tension for the cases analyzed.

(g)The vertical prestress is applied in calculation S10-0003 as a uniform pressure of 913 psi on the top edge of the model. The actual vertical prestress will not be a uniform pressure since the location of the vertical tendon is 6 inches eccentric of the centerline of the 42 inch thick wall. Based on a vertical tendon force of 1530 k and an uncracked gross wall section, the pressure distribution across the thickness of the wall due to vertical prestress would vary linearly from approx. 80 psi on the inside to 1814 psi on the outside. Therefore, the application of the vertical prestress load is not representative of the true load.

(h)The boundary conditions on the upper edge of the model that results in the deformed upper edge remaining horizontal under load is not representative of actual physical behavior since the wall is subject to non-uniform vertical deformation as well as bending under the vertical prestress load and detensioning forces.

(i)It is not clear how the bond (or lack of) at the interface between the conduit and concrete was modeled – Whether acting compositely or not? From the figure showing the model, many nodes at the interface appear to not coincide.

(j)The thickness of the conduit is considered to be 1/16”. The thickness of the conduit could also be 1/8”, which could impact the stress concentration around the conduit.

(k)Very limited analysis results have been documented in the calculation, which does not provide a complete understanding of the structural behavior. For example, no deformation plots and minimum principal stress plots for the three cases analyzed have been provided.

(l)The calculation results seem to contradict some of the discussion/conclusion in some of the failure mode evidence sheets (e.g. the “Discussion” section of FM 1.2, FM 7.3 conclusion, etc.)

Justify the use of the S10-0003 calculation in its current form, whose methodology, results and conclusion are questionable and subject to misuse, as supporting evidence in the root cause analysis. Also, address the issues raised in SIT Request #96.

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

Updated Response:

Calculation S10-0003 was done initially to determine if the absence of either the horizontal or compressive loads resulted in a more limited stress condition around the conduits than the case with both horizontal and compressive loads applied. The thinking was if a more limited stress condition was predicted, it would provide a basis for the detensioning sequence. In summary, this was a “what if” calculation to help work on the detensioning sequence. The calculation showed that the stresses with just a horizontal or vertical load were less than with both horizontal and vertical loads applied simultaneously; therefore it did not provide a basis for the detensioning sequence and was not used.

The calculation did have some graphics that illustrated stress concentrations around the tendons. PII used the calculation for illustrative purposes in some of the FMs because of this illustration. When the use of the calculation was questioned, PII substituted some of their own graphics and deleted references to the calculation. The calculation was not used for any technical purpose by PII as part of the root cause evaluation.

Based on feedback from reviewers, this MPR calculation is no longer included or used as a reference in the FMs or Root Cause Report. PII used their FEM analysis results as supporting documentation for the applicable FMs.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/3/2010

Status: Closed

Date Closed: 8/9/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 135

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: With reference to failure mode FM 7.5 “Added Stress Due to Removing Concrete at Opening,” and NRC Request #116, due to the limited scope of detensioning, the residual hoop and vertical prestress in the area of the SGR opening were not minimal. (a) What was the state of stress in and around the SGR opening (and specifically at the bottom right corner) at the time the cut was initiated at the bottom right corner with limited scope detensioning (b) Address any insights into the issue of destructive release of energy noted in NRC Request #116, and (c) Confirm if the series of figures with stresses from computer analysis in Exhibit 1 indicate stresses in the plane of the hoop tendons (delamination plane).

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #135

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 136

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Question

Request: Failure mode FM 7.5 “Added Stress Due to Removing Concrete at Opening,” document supporting evidence to be “none.” There are in fact supporting evidence for this FM such as: (1) concrete stress in the opening area due to prestress was not minimal due to limited scope detensioning when the cut was made (the concrete was in a semi-energized state). Also note that 4 or 5 of the hoop tendons near the top of the SGR opening were not yet detensioned at the time the cut was initiated. The limited detensioning could have thus created conditions for an impending failure by destructive release of prestressing energy; (2) The cut was initiated at a region of high stress/energy gradient (bottom right corner); (3) photographs indicate that water was observed gushing out of the crack to the right and below the opening near Buttress # 4 only on 10/2/09 and not on 9/30/09 when the opening cut was initiated; (4) Any differences observed during hydro-demolition between (a) when concrete was removed in the delaminated area for repair, versus (b) when concrete was removed for making the original SGR opening; (5) The fact that degraded (as opposed to as-found measured) concrete properties were required to be used in the finite element analysis to simulate the delamination by computer analysis (Ref. FM 7.1). Note that cores taken adjacent to the demolition area did not show degradation of strength properties (Refer Item c. under “Verified Refuting Evidence” section for FM 7.10).

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #136

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 137

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Question

Request: With reference to failure mode FM 7.5 “Added Stress Due to Removing Concrete at Opening,” the evaluation in Exhibit 2 is based on calculation S10-0003, which is questionable. Further, the boundary for concrete cut in refuting evidence #1 and EC 63016 Attachment Z09R21 (Exhibit 2 page 5) was not based on any explicit analysis of stress concentrations and peak tensile stresses. It is prudent engineering and construction practice to cut approximately mid-way between tendons rather than closer to tendons.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #137

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/22/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 138

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Please provide the PII document “Margin-To-Delamination Analysis of Proposed CR3 Detensioning Sequence Option 10F” referenced in FM 7.5.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #138

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 139

Individual Contacted: Charles Williams

Date Contacted: 5/24/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Please provide the report of the extensive PII FEM modeling details, material properties, inputs and results referred to in many root cause failure modes evaluations (e.g. FM 1.1, 1.15, 7.1, 7.3, 7.4, 7.5, etc).

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The detailed discussion is provided in the Root Cause Report.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 5/27/2010

Status: Closed

Date Closed: 6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 140

Individual Contacted: Charles Williams

Date Contacted: 6/3/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Please provide latest version (with exhibits) of FMs 1.1, 1.2, 1.15, 2.12, 3.4, 4.5, 7.1, 7.3, 7.4, 7.5 and 7.10.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Response located in L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\NRC Request #140

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 6/3/2010

Status: Closed

Date Closed: 6/15/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 141

Individual Contacted: Don Dyksterhouse

Date Contacted: 6/10/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: As stated in Section 4.1 of Calculation S10-0021, the radial pressure of 28.22 psi (taken from CR3 calculation S09-0054) is used for the design of radial reinforcing bars. Calculation S09-0054 documents other cases where the radial pressure is calculated taking into account the tendon duct void resulting in a higher radial pressure. The use of 28.22 psi in calculation S10-0021, which is the minimum radial pressure calculated in S09-0054, is not supported by any technical justification and is in contrary to the results of Calculation S09 0054 which documents other conditions that closely represent the physical condition of the containment wall. Further, the root cause investigation of the CR3 containment delamination has identified that significant tensile stress concentrations exist around the hoop tendon sleeves under fully tensioned tendon condition and was a contributing factor in the delamination. Please provide justification for the use of 28.22 psi radial pressure.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Per calculation S09-0054, Radial Pressure at Hoop Tendons, the radial pressure at the location of the hoop tendons is:

i) No area reduction: 28.12 psi
ii) Subtract void due to hoop tendon conduit: 38.69 psi
iii) Subtract void due to hoop and vertical tendon conduit: 45.46 psi

The radial pressure of 28.12 psi is based on an average center to center vertical spacing of hoop tendons of 19.22" (as shown on pages 6 and 8 of calculation S09-0054). The resulting force per inch width for design of radial ties is:

i) Assuming no area reduction for either hoop or vertical duct voids:
F1 = 28.12 psi x 19.22 inches = 540 lb/inch

ii) Effective area based on including area reduction due to tendon hoop voids:
F2 = 38.69 psi x (19.22-5.25) inches = 540 lb/inch

iii) Effective area based on including area reduction due to vertical and hoop tendon voids
F3 = 45.46 psi x {(19.22 -5.25) x (35.23 – 5.25)/35.23) inches} = 540 lb/inch

Based on the above results, the resulting force in the radial tie is the same for each of the three pressures because the force is based on the associated effective area. Therefore, the use of 28.22 psi in calculation S10-0021 is technically justified. The use of 28.22 psi instead of 28.12 psi is also slightly conservative.

A fully tensioned hoop tendon will result in relatively high, very localized radial tensile stresses in the concrete at the top and bottom of a hoop tendon that result in small vertical cracks in these regions. This phenomenon has been observed in the existing concrete at CR3 during concrete removal from Bay 3-4. These high, radial stresses are self- relieving due to the localized cracking and therefore do not contribute to the resulting force in the radial ties.

References:
S09-0054 Revision 0 Radial Pressure at Hoop Tendons (see request folder for reference)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/4/2010

Status: Closed

Date Closed: 10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 142

Individual Contacted: Don Dyksterhouse

Date Contacted: 6/10/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: CR-3 has had an operating experience of two delaminations (containment dome and cylinder) where soft/friable aggregate resulting in low concrete tensile strength was considered a contributing factor. Calculation S10-0021 implements a theoretical method of determining the capacity of the new grouted radial bars without regard to the site specific concrete properties. Additionally, an increased compressive strength of 5800 psi (vs. 5000 psi per the CR-3 current design basis) is used which further amplifies the theoretical capacity of the grouted anchor (it should be noted that the use of higher than 5000 psi compressive strength to increase concrete tensile capacity is the subject of another SIT question previously submitted). The maximum interaction ratio for the radial anchor design is 0.97 as shown on Page 17 of the subject calculation and this provides no tolerance for variability in the design input parameters used in this calculation. Considering the above and lack of site specific test data, the design methodology implemented in this calculation does not provide reasonable assurance of the capacity of the proposed anchoring system. Provide discussion relative to any planned testing of these anchors to ensure their capacity satisfies the design intent. Also, describe in detail the acceptance criteria and method of inspection for the grouting of these radial anchors.

Follow-up question:

Radial rebar pullout testing was performed in support of the Dome delamination repair. Has this data been reviewed and is it applicable to the radial bars in Bay 34?

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Conclusion summary: Testing performed at the mockup wall and testing performed at the wave protection paving structure provides reasonable assurance that the capacity of the radial bar anchoring system meets the design requirements identified in calculation S10-0021. The capacity of the 5/8" diameter radial rebars has been shown to be adequate thru a combination of ACI approved analytical methods and supported by test results. The analytical method applied a load factor of 1.5 to the calculated hoop tendon radial force and reduced the grout bond strength by a load factor of 1.67.

Radial rebar analysis discussion:
The grouted radial #5 rebar ties were evaluated for a ductile failure in calculation S10-0021 based on existing concrete having a compressive strength of 5800 psi (Ref. Calculation S10-0019), a hole diameter of 1 ½", an embedment of 12.5" and a load factor (LF) of 1.5 applied to the radial stress of 28 psi calculated at the hoop tendons. (Ref. Calculation S09-0054) Note that the load factor of 1.5 is conservative since per the FSAR the LF = 1.0 for tendon induced loads, primarily because the tendon prestress is carefully controlled during tendon tensioning. In the S10-0021 calculation, the characteristic bond stresses for Masterflow 928 grout between the grout and the reinforcing bar and the grout and the concrete were conservatively multiplied by a reduction factor of 0.6 to account for assumed cracks in the existing concrete. This is also a conservative assumption because the existing concrete in which the radial reinforcement is installed has been extensively inspected for cracks and per EC 75220 Attachment Z49, the average size of crack width for the 16 vertical cracks identified across the 3-4 bay (below El. 176') is 0.003 inches, i.e. extremely tight cracks. The reduction factor of 0.6 applied to the bond stresses in calculation S10-0021 is based on a maximum crack size of 0.012" >> 0.003" that exists in the area of

the radial bars installed below El. 176'.

Per calculation S10-0021, the interaction ratio for bond strength between grout to rebar is 0.9, and between grout to concrete = 0.97. A recent modification to the design increased the hole diameter of the radial rebar ties from 1 ½" to 2". This will further improve the bond strength between the grout and the surrounding concrete and provide additional capacity.

Radial Anchor Testing at the Mockup Wall:
Results: A total of 5 pullout tests were performed at the containment mockup wall. All 5 tests were successful and reached a tensile strength over 19 kips. Because the anchor was not tested to failure, the amount of force above 19 kips is not known.

Discussion: Pullout tests were performed on radial bars installed at the containment mockup wall. This wall was built using the 7000 psi mix design that will be used for the containment repair. Although this test was not performed in existing Crystal River 3 concrete, the test is valid to qualify the #5 rebar and the bond between the #5 rebar and surrounding grout. This is valid at the mockup structure and at the containment wall, because these items are identical at each location.

The allowable bond stress used in calculation S10-0021 for the grout to concrete interface, which represents the adhesion between the grout and the surrounding smooth surface of a drilled hole, is based on test results performed by the manufacturer in clean, saturated holes with f'c > 4000 psi. The type of aggregate used in the concrete is not a variable considered in the test results (Refer to Table 2, in Reference 10 of calculation S10-0021). Similar bond (adhesion) results are expected for the grout to concrete interface at both the mockup and the containment wall because the compressive strength of the concrete at both locations is greater than 4000 psi (7000 psi at the mockup and ≥ 5800 psi at the containment wall), and the holes for the radial bars were drilled and saturated for 24 hours prior to grout placement.

Capacity of Existing CR3 Concrete under direct Tensile Pullout based on Historical Test Data:
The following information is provided to give guidance on the expected strength of CR3 concrete when subjected to an anchor tension pullout load. The test results below are for wedge type anchor bolts, however, the overall tension load applied to the concrete is similar to the pullout force resulting from the radial rebar tests.

Test results from the 1983-84 "Concrete Anchor Qualification Program" (Reference 2) performed at CR3 in the Wave Protection Paving (WPP) along the south berm of the plant are shown below (results for Wej-It anchors are omitted due to their shallow embedment). The 28 day specified strength of the concrete at the WPP is 3000 psi (Reference 3 and 4). Per Reference 5, Section 3.08.5, concrete core samples were taken from each 100 square feet of surface in which the anchor tests were made. Per Reference 2, the results of the core compression tests were: 6940 psi, 7990 psi, 6750 psi and 7990 psi (Average = 7417 psi).

TABLE 1

1983 Test Results for 5/8" diameter Drillco Anchor Bolts in CR3 Concrete

Sheet#	Failure Load (lbs)	Ultimate Displacement (in.)	Embedment (in.) (After torqueing)
8	29800	0.5	7.625
9	29600	0.5	7.688
10	28600	0.5	7.688
11	30300	0.5	7.688
12	29500	0.5	7.500
AVERAGE	29560		7.6378

TABLE 2

1983 Test Results for 5/8" diameter Phillips Wedge Anchors in CR3 Concrete

Sheet #	Failure Load (lbs)	Ultimate Displacement (in.)	Embedment (in.) (After torqueing)
49	12500	0.25	5.938
50	15500	0.5	6.125
51	13600	0.25	5.938
52	14400	0.375	6.000
53	13600	0.500	6.000
54	16000	0.500	6.000
55	13200	0.500	6.000
AVERAGE	14114		6.000

Based on the test results contained in Tables 1 and 2 the deeper the steel embedment the greater the pullout resistance provided by the CR3 concrete. The Drillco anchors had the deepest average embedment = 7.6378" and the CR3 concrete provided adequate strength to resist up to an average pullout force of 29560 lbs.

Note that the force in the 5/8" radial anchors embedded a minimum of 12.5" = 10,860 lbs (Refer to Acceptance Criteria below) <<< 29,560 lbs tested capacity for the 5/8" diameter Drillco with only 7.6378" embedment.

Pullout strength of new radial anchors with minimum embedment = 12.5" per ACI 318-63:
The average resistance to a tension failure through shear cone pullout for the new 5/8" diameter radial anchors with minimum 12.5" embedment per ACI 318-63, Section 1707 is:

Radial anchor spacing per calculation S10-0021 is 17.5" x 19.5". Because shear cones will overlap with 12.5" embedment, calculate the tributary area to each anchor:
ATrib = 17.5 x 19.5 = 341 sq. ins.

P = 4Φ x (√f'c) x ATrib = 88.3 kips (f'c = 5800 psi, Φ = 0.85)

The tensile strength of a 5/8" rebar is:
FRod = Φ x Fy x As (Fy = 60 ksi, Φ = 0.9, As = 0.31 in2)

FRod = 16.74 kips <<< 88.3 kips therefore radial rebar will fail in tension prior to concrete cone pullout failure, i.e. ductile failure.

(see response folder for table)

Based on the above results, failure of the radial bars due to a concrete failure is not expected because the concrete capacity of a 5/8" rebar embedded 12.5" in 5800 psi concrete is 88,300 lbs (as calculated per ACI318-63) compared to the (minimum) test failure load of 14,114 lbs for a 5/8" Phillips wedge anchor embedded 6" in 7417 psi CR3 concrete. The 5/8" diameter radial rebar is predicted to fail per ACI 318-63 at 16,740 lbs, whereas actual testing showed the bars failing loads ≥ 19,000 lbs.

Acceptance criteria:
The actual radial load due to hoop tension is 16.286/1.5 (Ref. S10-0021, Section 5.1)= 10.86 kips. Since the radial stress is dependent on the force in the tendons, and the tendon prestress load is carefully controlled during tendon tensioning, the load factor can be considered = 1.0, similar to the load factor applied to prestress load combinations contained in the FSAR, Table 6.3. Therefore, any test result greater than 10.86 kips is acceptable. For conservatism, a load factor of 1.25 is considered. Therefore, any test load greater than 1.25 x 10.86 = 13.6 kips is acceptable, even though the acceptance criteria used for testing at the mockup wall was 19 kips (as discussed below).

The following acceptance criteria were used for the radial anchors installation (Reference 1):
1-Required inspection is performed and accepted by QC
2-Pull out tests are performed for the radial anchors installed in the mockup and test results revealed anchor's

capacity above 19 kips.
3-Installation tests are performed for the radial anchors to verify the flowability of the grout under specified temperature using ASTM C39. Tests were also conducted for different angles and different hole arrangement to verify that the grout is free from void and in full bond with the existing concrete. Acceptance of the installed radial anchors in the containment wall is subject of the acceptance of all installation tests.

Inspection criteria:
The following inspection activities will be performed during installation to ensure adequate installation of the anchors:
1-Verification of the hole diameter
2-Verification of the hole depth
3-Verification that the borehole is free from debris
4-Verification that the existing concrete in the borehole is pre-saturated
5-Verification that the borehole has no free water
6-Verification that the grout consistency and temperature are within the specified ranges
7-Inspection of the grout placement and verification of using "bird mouth"
8-Verification that the grout was placed using a tremie method from the bottom of the borehole to the surface.

References:
1.SGT Procedure for J-Hook Pull Test on Mock-up – Attachment 15 to WP 3732 Mock-up
2.MAR 82-07-26-01 "Concrete Anchor Qualification Program"
3.Drawing G-744-016
4.Drawing G-744-017
5.SP-5087

Follow-up response (11/4/10):

Per Section 5.2.7 of the Dome Delamination Report, testing of #6, Grade 60 radial reinforcement was performed in concrete blocks with a compressive strength of 2400 to 5000 psi. Each bar was embedded in a 15" deep hole and grouted with Masterflow 814 grout. The results of this testing indicated the development of a shallow "secondary failure cone" in the concrete at an average load of 31.5 kips. Failure occurred at an average load of 40.1 kip due to the rupture of the #6 bar at the weld for the testing apparatus attachments. (Note: not a concrete failure). Based on this information the following observations can be made:

1. Dome test specimens consisted of #6 rebar with 15" embedment. The new radial rebar for Bay 34 consists of #5 rebar with 12.5" embedment. Based on the failure load of 40.1 kips for the dome test, the equivalent failure considering only 12.5" embedment is 40.1 x (12.5/15)2 = 28 kips > 13.6 kips required for the new #5 reinforcing bars.

2. Dome testing was performed in concrete blocks with compressive strength of 2400 to 5000 psi. It is not known if the concrete blocks were produced from original CR3 concrete. The anchor bolt test results identified in this response were all performed in the wave protection paving which is original CR3 construction (28 day specified strength = 3000 psi < 5000 psi specified for the containment conservative).

3. The grout used for the dome radial anchors was Masterflow 814. The grout used for the new radial anchors is Masterflow 928. The bond properties between the rebar and surrounding grout may be different. However, Masterflow 814 grout was specified for this application and should be consistent with the containment concrete strength of 6000 psi. Grout cube compressive test results were between 4,265 and 7,215 psi.

4. The actual ultimate failure load is unknown. The Dome Delamination Report states that a shallow "secondary failure cone" developed at an average load of 31.5 kip for the eleven (11) tests. This is 3.3 kips higher than the minimum yield strength of the #6, Grade 60 bar. Tests proceeded up to an average failure point of 40.1 kips. Failure occurred in the bar at the attachment point to the testing apparatus. Thus indicating that the cone failure was localized and may not have impacted the overall tension capacity of the embedded #6 rebar. Since the tests

were stopped when the reinforcing steel broke, the ultimate failure load for 15 inch embedment is unknown. The anchor bolt testing done at the wave protection paving was performed to failure which consisted of either concrete failure, anchor failure or anchor elongation reaching 0.5” as prescribed in ASTM E488, Section 10.

In summary, the test results support conclusions applied in the design of radial reinforcement in Bay 34. Since testing of #6 Grade 60 radial reinforcement embedment capacity for the dome repair was not taken to concrete failure, it cannot be used as a prediction of ultimate strength. However, the results exceed values used in design for #5 radial reinforcement bars by a large margin. The safety factor is greater than 2 using this result alone.

Misc Notes:

Response By: Don Dykstehouse

Reviewed By:

Date Response Provided: 8/20/2010

Status: Closed

Date Closed: 11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number:	143		
Individual Contacted:	Don Dyksterhouse	**Date Contacted:**	6/10/2010
Requestor/Inspector:	George Thomas	**Category:**	Information Request

Request: According to Section 4.4 of Calculation S10-0021, it is assumed that no shear stress will be resisted by the radial bars. The justification given in this section is not sufficient to disregard the effects of combined shear and tension stresses. Provide detail justification, in a quantitative form, for the exclusion of shear stresses from the design of new anchors when radial and plane shear stresses in the area of the new grouted anchors are expected due to the applicable design basis loading combinations.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

For areas that require radial anchors, the nominal as-built excavation thickness is 15". The radial tension force per Calculation S09-0054, Radial Pressure at Hoop Tendons, is located in a plane between the centerline of the hoop tendon and the inner face of the hoop tendon. Because the inner face of the hoop tendon is at 12.375" from the outer face of the wall, the shear at the radial tension plane will occur within the new concrete. The new concrete has sufficient capacity, as shown below, to resist both radial tension and shear force in the section.

Shear Capacity: (ACI 318-63, Section 1701-c)

(see request folder for formula)

The maximum shear stress in the wall due to a factored load combination is 37.5 psi as determined in Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall. (Load Combination 5 – Cut # 1 – Elevation 239.3')

The allowable tensile capacity of the new concrete for membrane stresses, according to CR3 FSAR– Section 5.2.3.3.1, is equal to

(see request folder for formula)

This value is significantly higher than the radial stress estimated in Calculation S09-0054 (28 psi).

The interface between existing and new concrete at a depth of 15 inches is addressed in Calculation S10-0060, Evaluation of Shear Stresses on Circumferential Interface of Existing to New Concrete in Bay 34. The calculation concludes that adequate clamping force exists to ensure shear capacity. The capacity exists because of radial compression at the bond interface from hoop tendons. No credit is taken for the clamping force offered by the radial bars.

Notice that due to the nature of the excavation (42" to 24" to 15"), an interlock between the new and the existing concrete is developed. A shear failure across this interface will not occur because the new concrete will not have a displacement relative to the existing concrete.

In summary, there is no shear demand on the radial bars. The radial bars do not contribute to shear-friction capacity. Therefore, design of radial bars to resist radial forces associated with tendon curvature alone is adequate.

REFERENCES:
1.Calculation S09-0054 Revision 0, Radial Pressure at Hoop Tendons
2.Calculation S10-0060, Evaluation of Shear Stresses on Circumferential Interface of Existing to New Concrete in Bay 34 (not approved)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/4/2010

Status: Closed

Date Closed: 11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 144

Individual Contacted: Don Dyksterhouse

Date Contacted: 6/10/2010

Requestor/Inspector: George Thomas

Category:

Request: The new anchors are installed at the interface of old and new concrete. Provide information, in a quantitative form, relative to the shear and tension loads that the new anchors may be subjected to due to the loading conditions where bending of the shell is expected (e.g., accident thermal loading).

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

As discussed in response to Question 143, Calculation S10-0060, "Evaluation of Shear Stresses on Circumferential Interface of Existing to New Concrete in Bay 34" evaluates the shear and radial compression that exists on the interface between old and new concrete. The analysis concludes that adequate clamping force exists to ensure shear transfer. Shear demand is determined by finite element analysis and includes the effect of shell bending, where appropriate. Note: The supplemental response to Question 145 addresses further analysis for the effect of beam shear from thermal gradient. A thermal gradient was not included in the finite element analysis because it has no effect on the shear.

In summary, there is no shear demand on the radial bars. The radial bars do not contribute to shear-friction capacity. Shear associated with shell bending is transferred through concrete shear capacity for the zone of concrete radial tension. Where concrete is in radial compression (where the bond interface exists), shear, including the effect of shell bending, is transferred through shear-friction with no reliance on radial reinforcing bars.

REFERENCES:
1. Calculation S10-0060, "Evaluation of Shear Stresses on Circumferential Interface of Existing to New Concrete in Bay 34" (not approved).
2. Response to S.I.T. Question 143.
3. Supplemental Response to S.I.T. Question 145.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 10/5/2010

Status: Closed

Date Closed: 11/10/2010

Request Number: 145

Individual Contacted: Don Dyksterhouse

Date Contacted: 6/10/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Calculation S10-0021 states that the new anchors will be installed where a nominal 14" of concrete has been removed. This implies that there is no plan to install anchors where a nominal of 24" concrete is removed. Provide information, in a quantitative form, the adequacy of the interface of old and new concrete (to act as a monolithic 42" thick wall) to resist the expected stresses (under all applicable design basis loading conditions, prestressing, seismic, thermal, pressure, etc.) without any anchors.

Follow-up request received verbally from the NRC on 8/25:

1) Document the analyses associated with this question resolution in an updated formal calculation
2) Demonstrate that we meet the original Code of Record, ACI 318-63, specifically Chapter 25 as it applies to this situation.
   a. Document basis for engineering judgment when applied
   b. Address what safety margin exists for each service and factored load combination.
   c. Account for uncertainty in properties to ensure "sufficient margins" beyond code allowables to account for the unique CR3 situation:
      i. Site specific aggregate,
      ii. The effect of creep that would cause additional loads,
      iii. The effect of shrinkage that would cause additional loads.
3) Document how adequate surface preparation was determined and ensured in the containment work.
4) Instrumentation to monitor interface - latest version of EC that NRR reviewed only showed one strain gage in Bay 3-4, although it was discussed that there are 5 currently in the EC. Request was to include appropriate instrumentation at the interface to detect movement and ensure redundancy so that not relying on a single instrument. Monitoring of instrumentation should be performed during the tendon tensioning evolution and during pressure tests that follow restoration of containment integrity.

Follow up question received from Meena Khanna on 10/18/10:

In response to Item (4) of the supplementary question on Request #145 regarding instrumentation to monitor the interface of new and old concrete in the 24" excavation area, the response indicates that there are embedded strain sensors installed in the new concrete in the middle mat area close to the surface of the original concrete layer. It is not clear how the instrumentations will provide information regarding the relative movement of the interface between the old and new concrete in the plane of the wall and normal to the plane of the wall, since they do not cross the interface. In addition, the old and new concrete, the new rebar, and the instrumentation installed in the new concrete will register the appropriate strain level, due to the action of tendon retensioning and pressure during the SIT. Please discuss how this strain will be differentiated from the strain caused by potential slippage or separation between the two layers of concrete.

Please describe the physical characteristics of the proposed instrumentation and discuss how your proposed instrumentation will be capable of providing relevant information related to the in-plane and radial movement at the interface between the old and new concrete.

In addition, the response to Item 4 states the following:

"The trend of recorded strains will positively identify any sudden change that is sufficiently large to be of structural significance."

This statement is not acceptable to the staff. The interface between the old and new concrete has no mechanical anchors and relies on the bond between the old and new concrete and the compression applied by the hoop tendons to maintain its integrity in the plane of the wall and normal to the plane of the wall. Any slippage or separation at this interface, recorded during the retensioning or the SIT, is not an acceptable design condition and it invalidates the design basis calculations performed for CR3.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

1.0 Radial Tension about Hoop Tendons:
As determined in Calculation S09-0054, "Radial Pressure at Hoop Tendons", the maximum radial tension stress = 28.12 psi based on the radius to the inner most point of the conduit as the effective radius for the hoop tendon. The maximum radial tensile stress will be resisted by #5 single leg and closed leg ties that are provided in the new nominal 24" thick concrete and which are fully developed about the hoop tendons (Refer to sketch page 7 and Ref. 1, Section 6.3.1.2).

For the 24" removal area (Elevation 176' to 231'-6"- Refer to sketch on Page 5), the joint between the new and existing concrete inboard of the hoop tendons is under constant radial compression that will resist the vertical and horizontal shear flow forces, no radial anchors are required across this joint. The maximum average vertical and horizontal radial shear forces (Refer to sketch on Page 6 for orientation of shear forces) resulting from all design basis load cases are obtained from Reference 1 and are evaluated in Sections 2.0 and 3.0 of this Response.

2.0 Radial Shear (Horizontal Plane) & Resulting Vertical Shear Flow at Interface of Old to New Concrete:
Per Reference 1, Section 6.3.1.1 and Appendix D, the maximum average horizontal radial shear force = 12.35 kip/ft and is associated with Load combination 4 (0.95D+Fa+Ex+Ev+P+Ta) – Cut #6 at Elevation 176.3' (Note that the 24" excavation extends from radius 13.3' to 24.9' feet as shown in Table 6-1and Figure 6-10 of Ref. 1). This transverse shear force of 12.35 kip/ft will result in simultaneous and equal longitudinal and transverse shear stresses at any point across the thickness of the section under consideration. The transverse and longitudinal shear stresses at a plane through the thickness of the section (shear flow) across plane a-a is as follows:

q = VQ/It

(see response folder for figure)

V = 12.35 kips
Y = 24 – (42/2) = 3"
Q = 12 x (42-24) x {3 + (42-24)/2)} = 2592 in3
Icg = 12 x 423/12 = 74088 in4
 qa-a = (12350 x 2592)/(74088 x 12) = 36 psi

3.0 Radial Shear (Vertical Plane) & Resulting Horizontal Shear Flow at Interface of Old to New Concrete:
Per Reference 1, Section 6.4.1.1 the maximum vertical face radial shear force = 7.09 kips/ft and is associated with Load Combination 1 (0.95D + Fa + 1.5P + Ta) - Cut #18. The corresponding transverse and longitudinal

shear stresses are:

(36/12.35) x 7.09 = 20.67 psi

4.0 Total Compressive Stress at Interface of Old to new Concrete:
The total compressive force at the interface between the old and new concrete is based on two primary component forces as described below:

1. Due to the prestress in the hoop tendons the portion of the wall inboard of the hoop tendons is in compression in the radial direction. The maximum radial compressive stress (pressure) developed by a hoop tendon based on minimum end of life tendon force is:

Tendon Force FT = 1435 kips (Ref. 1)
Radius to centerline of hoop tendon ri = 65.00 ft = 812.625 in (Ref. 1)
Hoop tendon ring mean spacing = 19.3 in (Ref. 1)
Wall thickness tw = 42 in
Tendon force per linear foot = 1435 x 12/(19.3) = 929.5 kips/foot
Resulting compressive pressure PHoop = 929.5/ (12 x 812.625) = 95.32 psi

2. The controlling load case for vertical shear flow is 0.95D+Fa+Ex+Ev+P+Ta (Section 1.0) and for horizontal shear flow 0.95D + Fa + 1.5P + Ta (Section 3.0). The accident pressure component of each load case results in a compressive stress on the interface joint of:

(see response folder for figure)

1.0P

The compressive stress due to the accident pressure (1.0P) at the 24" location is (55 x 24)/42 = 31.43 psi. Note that the load factor of 1.5 is not considered when calculating the compressive resistance.

(see response folder for figure)

1.0P or 1.5P + Radial Hoop Compressive Stress

*Conservatively assumed location of maximum resulting radial compressive pressure (centerline of a hoop tendon)

**Resulting compressive force due to hoop tendon prestress of 95.32 psi at a depth of 24" P24 = {67.2 x (18)} / (42-9.75) = 37.5 psi

Total compressive force at interface between old and new concrete due to the two controlling load cases is:

(0.95D+Fa+Ex+Ev+P+Ta): 37.5 + 31.43 = 68.93 psi

(0.95D + Fa + 1.5P + Ta): 37.5 + 31.43 = 68.93 psi (Only 1.0P considered in calculating compression)

5.0 Resistance to Shear Flow at Interface between New to Old Concrete:
Conservatively assuming the interface between the new and old concrete were to de-bond, the two sections will not separate due to the compressive pressure load of either 68.93 or 84.64 psi at the joint between the old and new concrete. Use a coefficient of friction = 1.0 (ACI 318-05, Section 11.7.4.3) then the vertical and horizontal shear resistance is:

RSF (0.95D+Fa+Ex+Ev+P+Ta) = 1.0 x 68.93 = 68.93 psi > 36 psi OK

RSF (0.95D + Fa + 1.5P + Ta) = 1.0 x 68.93 = 68.93 psi > 20.67 psi OK

Conclusion:
The 42” thick wall is under compression at the joint between the new and old concrete due to hoop tendon prestress and accident pressure (typical for all controlling accident load combinations). It has been shown that the resulting vertical and horizontal shear resistance due to friction (both for prestress and prestress +accident pressure) at the joint interface is greater than the applied shear stresses (shear flow) for the load combinations that produce maximum radial stresses.

The resulting maximum vertical shear flow stress of 36 psi and the maximum horizontal shear flow stress of 20.67 psi result from different load cases and occur at different locations on the repaired wall. Therefore, combining these results by the SRSS method is not relevant.

Based on the results shown in this Response the interface between the new and old concrete will be able to resist the expected stresses resulting from all design basis loads.

REFERENCES (see response folder for references):

1.Calculation S10-0030, Revision 0, Reinforcement Design for Delaminated Containment Wall
2.Calculation S10-0012, Revision 1, Stresses Around the SGR Opening due to Design Basis Loads

Supplemental Response 1:

SIT Question 145 requests justification for the capacity of the old to new concrete interface to resist design loads in light of the fact that radial reinforcing bars are not being used at the joint. The interface of interest is the area to the left and right of the SGR opening where 24 inches of concrete was removed leaving a thickness of about 18 inches.

The response to SIT Question 145 compiled on 8/13/2010 compares shear stresses at the old to new concrete interface with the available compression. The comparison shows that adequate capacity exists due to shear-friction as described in ACI 318 Section 11.7. In ACI 318, shear-friction is made possible by clamping that occurs due to tension on reinforcing steel when adequate displacement is allowed to occur. Clamping at the interface of old to new concrete is provided by radial compression that exists due to curvature of hoop tendons. In the response to SIT Question 145, the shear demand was approximated (conservatively) from existing calculations without performing further computer manipulations. The available shear values were those values used to check shear capacity of the new concrete on the horizontal and vertical planes. The values were converted to shear flow for the plane of interest. Since the values were less than the available compressive forces and a shear-friction coefficient of 1.0 was considered reasonable per ACI, further analysis was not performed.

During a subsequent conference call with the NRC on 8/17/2010, follow-up discussion occurred based on the SIT Question response. The discussions focused on the impact of other possible loading conditions. These loads included the effect of a thermal gradient, the effect of in-plane tension where the new and old concrete have different moduli, and the effect of creep. Additionally, because limestone aggregate used in the concrete at CR3 was identified as a contributor to the root cause of delamination, the affect that limestone aggregate may have on the shear capacity was discussed. As a result, the selection of a shear-friction coefficient of 1.0 was questioned. Suggestions have been made that further testing to determine the shear-friction parameters may be warranted or that additional reinforcing steel may need to be placed at the interface.

In response to the discussion, additional evaluations have been made for both the determination of loads and the shear-friction capacity.

Thermal Loads:

An ANSYS finite element run had been previously made for the effect of the normal operating thermal gradient. The gradient is 37 degrees on the outside and 99 degrees on the inside surface of the concrete wall. The computer run was post-processed to determine shear stresses due to thermal gradient at the 24 inch depth. The stresses from the ANSYS model were essentially zero. This result is reasonable because the effect of the thermal gradient is a relatively constant moment. With no moment variation, there is no general shear and therefore, no beam shear.

In-Plane Forces:

To address the possibility of an impact from hoop or vertical tension to materials with different elastic moduli, the other load cases were also post-processed to generate shear stresses at the 24 inch depth. This is an alternate and more rigorous way to determine shear stresses compared to what was reported in the response to Question 145. The load cases addressed were dead load plus prestress alone and with the design combinations that include LOCA and seismic. The vector sum of the hoop and vertical shear stresses was determined for comparison to the radial compression. The average radial compression on the plane of interest was also extracted for these load cases. Comparing shear demand to the available radial compression clamping force indicates that the highest shear-friction coefficient needed is 0.45. This occurs for the combination that includes only dead load plus prestress. Other cases that include accident pressure require a friction coefficient of about 0.2. The case with seismic alone requires a coefficient of 0.4. Minimum required values derived from this sensitivity assessment are less than the recommended value from ACI 318 Section 11.7 and therefore acceptable. ACI recommends a coefficient of 1.0 for concrete placed against hardened concrete with the surface intentionally roughened. Higher values can also be justified because the shear stresses are very low. Based on the stresses extracted from the ANSYS model, the shear demand at the interface of new to old concrete is less than 30 psi for all load cases.

Aggregate Shear Capacity:

The effect of aggregate on the shear capacity of the joint is minimized by the conservative assumption of shear-friction per ACI Section 11.7. For analysis purposes, the surface is assumed to have failed and no credit is taken for the concrete in shear. Section capacity is a function of the mechanical interlock that exists on the surface. It should be noted that the concrete surface was intentionally roughened to CSP-9 conditions (full amplitude of at least ¼ inch).

One way to assess the effect of soft or friable aggregate on the shear-friction capacity is by a simple volume ratio. Reference is made to the GAI Report 1913 for this purpose (See Appendix C "Final Report of Concrete Quality Evaluation"). Page C-16 lists the friable particles to be 0.24% and soft particles to be 9.98%. Specific gravity and absorption are also listed as 2.41 and 4.70%, respectively. For the purpose of this check, the specific gravity is conservatively assumed to be saturated surface dried (SSD) conditions. The mix typically includes 1800 pounds per cubic yard (SSD) of coarse aggregate. Therefore, the volume of soft and friable aggregate would be 10.22% (1800)/(2.41*62.4)=1.22 cf/cy or 4.53%. Assuming these particles have minimal strength properties, a conservative reduction for shear-friction capacity would be about 5%.

Creep is not expected to have a significant effect because the mix design for the new concrete was developed for this specific purpose. The creep coefficient from 90 day testing is 1.1. The creep coefficient for the old concrete was determined to be 1.6 and because it has been in a relaxed state for an extended period, creep recovery is about 28% of the elastic strain (ACI 209.2R-08 cites two references on creep recovery. Application of the creep recovery function with about 6 months of creep recovery yields recovery of 28% of elastic strain). Since the new concrete is slightly stiffer, it will initially have more stress. The ratio is approximately equal to the ratio of the moduli of elasticity. Since the creep coefficient of the new concrete is higher than the recovered creep for the old concrete, the stress becomes more balanced. To determine the long term effect of creep, strain compatibility can be used.

$$\frac{\sigma_1}{E_1}(1+v_1)=\frac{\sigma_2}{E_2}(1+v_2)$$

$\sigma 1 = E1/E2 * (1+v2)/(1+v1) * \sigma 2$

$\sigma 1 = 3.6/6*(1+1.1)/(1+0.28) = 0.98*\sigma_2$

Therefore, the long term difference in stress due to different modulus values and different creep is less than 5%. This assessment uses best estimate values for modulus of elasticity. It also makes the assumption that the old concrete will not creep beyond the recovered creep. This assumption is justified by the fact that the old concrete is more than 33 years old. In summary, the final stresses after creep will be essentially balanced between the old and new concrete.

Tests to Evaluate Coefficient of Friction:

An informal test was performed to approximate the shear-friction coefficient. The test was performed on a core extracted at an existing vertical crack. One half of the core was stabilized on a flat surface. A load was applied to the top half at a 45 degree angle to approximate the equivalent normal and lateral forces. No sliding occurred due to load applied at 45 degrees. Further testing was performed by applying a constant load and changing the angle until slippage occurred. The angle was monitored with a digital angle finder. The test was repeated 3 times and the slippage occurred at an angle of 18 degrees from horizontal (average of 3 tests). This angle equates to a shear-friction coefficient of 3.2, significantly higher than the shear-friction coefficient recommended in ACI 318 Section 11.7 that was initially used.

(see response folder for diagram)

Existing tests are also discussed in a PCI Journal article from May-June 2008 "Calculating shear friction using an effective coefficient of friction" by John A. Tanner. The recommended effective shear-friction coefficient is 2.9 for this application (low shear, roughened normal weight concrete).

Confirmatory tests with site-specific concrete are also planned. Testing will implement the methods described in ASTM D-5607, "Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force". Our testing will be limited due to the intended application in this case. The effective shear-friction coefficient is the parameter of interest. In summary, the test is performed by maintaining a constant force normal to the nominal shear plane of the specimen and increasing external shear force along the designated shear plane. The applied normal and shear forces are recorded and used in determination of the required parameters.

Summary:

Use of shear-friction with clamping force associated with hoop tendon compression is more effective to address shear stress than use of radial reinforcing steel because the clamping force from prestress exists at this plane for all design conditions without any required displacement. Placement of reinforcing steel could be done but would not contribute to shear-friction. Conservative values have been used to validate that shear-friction capacity exceeds the shear demand at the interface of new to old concrete.

References:
1GAI Report 1913 – "Final Report - Reactor Building Dome Delamination – December 10, 1976"
2PCI Journal May-June 2008, "Calculating shear friction using on effective coefficient of friction"

Supplemental Response 2 (10/5/10):

Item 1:
Calculation S10-0060, Evaluation of Shear Stresses on Circumferential Interface of Existing to New Concrete in Bay 34, is being finalized to address shear demand versus shear-friction capacity at the interface plane between the existing and the new concrete.

Item 2:
ACI 318-63 Shear Capacity
The code of record for CR3 is ACI 318-63. The term, shear-friction, is not included in the 1963 code. Shear-friction was introduced by this name in the 1971 edition. However, the concept is discussed in ACI 318-63 Section 2505 regarding shear transfer along the interface between the flange and web in composite construction. In that application, the web is typically a precast beam and the flange is a reinforced concrete slab. The two sections can be tied together with anchors. However, shear capacity is assigned even without anchorage. ACI 318-63 Chapter 25 "Composite Concrete Flexural Construction" Section 2505 allows 40 psi of bond capacity for rough and clean contact surfaces under service loads. If the minimum reinforcing steel (0.15%) is added, the bond capacity for service loads can be increased to 160 psi. Service load capacities are increased by 1.9 for ultimate loads. The requirement to use capacity reduction factors is not addressed in ACI 318-63 Chapter 25. Without a capacity reduction factor, the ultimate load capacities would be 1.9*40 psi=76 psi for a clean rough surface with no ties and 1.9*160 psi=304 psi when ties are used in combination with a clean rough surface.

In ACI 318-1971, the composite concrete flexural member chapter (17) was updated to allow 80 psi and 350 psi for these two cases. However, ACI 318-71 requires a capacity reduction factor. Per ACI 318-71, the values would be 80 psi*0.85=68 psi and 350*0.85 psi=298 psi. These values are slightly less than allowed by the ACI 318-63 code for ultimate load capacities. The most conservative interpretation of both editions of the code is implemented for this application. For service loads, 40 psi and 160 psi are used. For ultimate loads, a capacity reduction factor is applied to the ACI 318-63 capacities discussed above.

Shear stresses at the 24" excavation depth plane:
Calculation S10-0060 provides a summary of shear stresses on the circumferential plane at the nominal 24 inch excavation depth. S10-0060 Attachment 2, Appendix E shows average stresses for areas that are about 7 feet wide and about 6 feet tall. Averaging was used to ensure that the stresses were not due to local effects. This area is less than about 3 times the thickness. The highest shear stress for normal load combinations is about 35.9 psi (compared to 40 psi allowable). For factored load combinations, the highest shear stress is about 30.7 psi (compared to 64 psi allowable). The safety factors are 1.1 and 2.1 for normal and factored combinations, respectively. Shear stresses near the center of the 24 inch area are lower.

Additional analysis has been performed to demonstrate the conservatism of comparisons made using the 1963 code. For code editions later than 1963, shear-friction is specifically addressed. Clamping action is multiplied by a friction coefficient to determine the available shear capacity. Significant additional margin is available with the 1971 provisions. The discussion in the later code also reveals the mechanism involved in the provisions. This allows for site specific material properties to be confirmed against the provisions. S10-0060 Attachment 2, Appendix F provides a comparison of shear stress to radial compression. Values in the table are equivalent friction coefficients that would be needed to ensure no movement. The highest required coefficient of friction value from the table is 0.8 for normal operating load combinations and 0.3 for factored load combinations. Tests with Crystal River site specific concrete with Florida aggregate have demonstrated that the actual friction coefficient is about 1.4. This friction coefficient results in safety factors of 1.75 and 4.67 for normal and factored combinations, respectively.

Creep:
The effect of creep was addressed in Supplement 1 to the Q145 response. In summary, the final stresses after creep will be essentially balanced between the old and new concrete.

Additionally, creep is self-limiting. Any movement along the old – new concrete interface relieves potential stresses caused by differential creep without causing an impact to shear capacity. Shear capacity at the interface is dependent on shear-friction which is effective with cracked concrete.

Shrinkage:
Placement of new concrete against the existing concrete will not create any meaningful shrinkage stresses. The shrinkage front from the new concrete will not reach the interface for many years, and by then the interface will

be in moisture equilibrium. It will be impossible to detect the interface from moisture readings alone. The attached article on containment concrete addresses moisture content results from forced drying for about 30 years. The paper provides an indication of relative humidity with depth. At Crystal River, an average ambient relative humidity is about 73%. Based on this paper, the relative humidity would be expected to be near 100% at the depth of the interface of old to new concrete even after 30 years.

Item 3:
Surface preparation is addressed in EC 75219. The EC requires that concrete surfaces have a minimum surface profile (profile height) of not less than CSP-9 (heavy scarification) (reference International Concrete Repair Institute - ICRI 03732 "Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays" January 1997). Surface profiles were verified by QC using plastic model concrete surface plaques as discussed ICRI 03732. Surface preparation testing is also performed with ICRI 03739 "Guide to Using In-Situ Tensile Pull-off Tests to Evaluate Bond of Concrete Surface Materials" March 2004, as a guideline. Bond at the surface was tested to ensure against degradation (bruising) due to concrete removal techniques.

Item 4:
Three groups of embedded strain sensors (No. 4 hooked bars instrumented with strain gages and proprietary concrete embedment gages) are installed in the new concrete in the middle mat area close to the surface of the 18 inch original concrete layer. Each group includes instrumented bars oriented in both the vertical and hoop directions. One group includes an instrumented bar inclined at 45° to the hoop / vertical directions and one includes hoop and vertical proprietary (Micro-Measurements product) embedment gages that serve to back up the correspondingly oriented instrumented bars. Any shear failure across the old to new concrete interface will result in a sudden change in new concrete strain. This will be sensed by the embedded instruments and recorded. The trend of recorded strains will positively identify any sudden change that is sufficiently large to be of structural significance.

In addition, 6 proprietary embedment gages are installed across the interface between original concrete and the new concrete. One of these is located in the (nominal) 24 inch excavation area. The remaining five are in the (nominal) 10 inch excavation area. These sensors are intended principally to detect radial separation between the old and new concrete layers but will also respond to shear movement between the layers.
In aggregate, these multiple instruments provide a sufficient amount of redundancy to monitor Bay 34 physical responses.

REFERENCE:
1. Calculation S10-0060, Evaluation of Shear Stresses on Circumferential Interface of Existing to New Concrete in Bay 34, (Not approved).
2. Engineering Change - EC 75219 Reactor Building Delamination Repair- Phase 3 Concrete Removal.
3. International Concrete Repair Institute - ICRI 03732 "Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays" January 1997.
4. International Concrete Repair Institute - ICRI 03739 "Guide to Using In-Situ Tensile Pull-off Tests to Evaluate Bond of Concrete Surface Materials" March 2004.

ATTACHMENT:
1. Nilsson, L.-O. and P. Johansson, J. Phys. IV France 136 (2006) 141–150, "The moisture conditions of nuclear reactor concrete containment walls – an example for a BWR reactor" (see request folder for attachment)

Supplemental Response 3 (10/28/10):

This third supplemental response to S.I.T. Question 145 addresses two follow-up items. First, Supplement 2 was provided to address questions discussed during a conference call on August 25, 2010. Response Item 1 in that supplement discusses a calculation that was not finalized at the time of submittal of the response. That document is now approved and summarized on the attached page.

Secondly, a follow-up question was provided by the NRC on October 18, 2010 as follows:

In response to Item (4) of the supplementary question on Request #145 regarding instrumentation to monitor the interface of new and old concrete in the 24" excavation area, the response indicates that there are embedded strain sensors installed in the new concrete in the middle mat area close to the surface of the original concrete layer. It is not clear how the instrumentations will provide information regarding the relative movement of the interface between the old and new concrete in the plane of the wall and normal to the plane of the wall, since they do not cross the interface. In addition, the old and new concrete, the new rebar, and the instrumentation installed in the new concrete will register the appropriate strain level, due to the action of tendon retensioning and pressure during the SIT. Please discuss how this strain will be differentiated from the strain caused by potential slippage or separation between the two layers of concrete.

Please describe the physical characteristics of the proposed instrumentation and discuss how your proposed instrumentation will be capable of providing relevant information related to the in-plane and radial movement at the interface between the old and new concrete.

In addition, the response to Item 4 states the following:

"The trend of recorded strains will positively identify any sudden change that is sufficiently large to be of structural significance."

This statement is not acceptable to the staff. The interface between the old and new concrete has no mechanical anchors and relies on the bond between the old and new concrete and the compression applied by the hoop tendons to maintain its integrity in the plane of the wall and normal to the plane of the wall. Any slippage or separation at this interface, recorded during the retensioning or the SIT, is not an acceptable design condition and it invalidates the design basis calculations performed for CR3.

Response:
EC 75220 Attachment G02R23 "SK-75220-C002 - Instrument Location Map" identifies the locations of sensors, including four radial gages that were added in the 15 inch excavation depth in early August (after the question was asked).

In summary, one radial sensor is located across the new to old concrete interface at the 24 inch depth. At three locations near the interface (middle mat depth) an array of strain gages is provided to monitor shear strain in the new concrete. These sensors are located between 2 and 6 inches from the interface plane. Additionally, two of the new radial gages at the 15 inch depth are located within 18 inches from the transition between the 15 inch depth and the 24 inch depth. These are located to the left and right of the 24 inch excavation area. The other two are located at the 15 inch depth just above the 24 inch excavation area. One is about 2 feet above and the other is about 6 feet above the transition between 15 inch and 24 in excavation.

The interface between the original and new concrete in the 24 inch excavation area is in radial compression under the pre-stressing load as well as all other design loads. Therefore, radial separation is not considered possible and any relative movement between new and original concrete must be in the hoop-vertical plane. Regardless, postulated radial separation can be detected by redundant strain gages at this depth. As discussed below, strain in the circumferential plane will be recorded adjacent to this postulated plane of separation in the hoop at three locations with hoop and vertical gages and one diagonal gage when tendons are tensioned. There are a total of nine strain instruments at these three locations.

Friction due to radial compression, as well as bond, at the rough surface interface eliminates the possibility of relative movement between concrete layers unless such movement is accompanied by the release of shear stress across the interface. When shear stress releases, static equilibrium requires that direct stresses, and corresponding strains, in the contacting layers change. This change in vertical and/or hoop strain will be reflected as a sudden change in the output of the strain sensing instruments installed in the new concrete. This approach for monitoring movement along a postulated zone of discontinuity is considered to be the most appropriate and reliable method. Typical strain instruments located across the interface will be damaged if movement was to occur. Anomalous movement would not be distinguishable from random failure of the device. With shear gages located in the new concrete adjacent to the proposed zone of discontinuity, verification of instrument operability can be performed after the postulated slippage event. The radial gage installed at the 24 inch depth will not provide meaningful information if delamination were to occur. Such an event would have to be confirmed with gages located near the interface on the circumferential plane.

Additionally, radial gages are located around the perimeter of the 24 inch excavation area. New concrete within this perimeter is nominally 24 inch thick. The existing concrete within this perimeter is nominally 18 inch thick. The width of these panels is about 12 feet. Postulated radial separation at the interface would be observed at the perimeter because the panels freed by the separation are very stiff. These new instruments are installed to monitor radial strain for confirmation of performance of the radial anchors. If the objective had been to detect delamination, a different arrangement would have been selected. However, the instruments will provide useful indication if such a separation were to occur.

If sudden changes in strain are detected during tendon tensioning or containment pressure testing, the impact and extent of any slippage or radial movement on the integrity of the interface will be evaluated against CR3 design basis requirements.

Regarding the statement:

The trend of recorded strains will positively identify any sudden change that is sufficiently large to be of structural significance.

The intent was not to imply that the design employs or expects slippage or separation. However, one intent of the statement was to recognize the presence of micro-cracking that will not be detected by these instruments. Additionally, as required, the design will fully comply with the code of record and design basis if any anomalies are detected.

In summary, strain gages located as indicated on the attached drawing will be used to record movements during tensioning and pressure testing. Spacing and redundancy is considered to be adequate for monitoring of the interface. Slippage or radial separation readings can be differentiated from other readings by the sudden nature of the movement compared to the gradual and predictable movements associated with tendon tensioning and pressure loading. Evaluation of anomalous results will ensure compliance with CR3 design basis requirements.

REFERENCE:
1.Engineering Change - EC 75220 Reactor Building Delamination Repair- Phase 4 Concrete Placement.

ATTACHMENT: (see request folder for attachments)

1.Summary of Calculation S10-0060 – Evaluation of Shear Stresses on Circumferential Interface of Existing to New Concrete in Bay 34
2.EC 75220 Attachment G02R23 “SK-75220-C002 - Instrument Location Map”

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/12/2010

Status: Closed

Date Closed: 11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 146

Individual Contacted: Don Dyksterhouse

Date Contacted: 6/10/2010

Requestor/Inspector: George Thomas

Category: Information Request

Request: Consistent with the dome delamination radial anchor repair (performed in 1976), provide a commitment that sufficient strain instrumentation will be installed for the radial anchors to ensure that the structure and the radial anchors are responding, during the post-repair test, as designed.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Strain gages for the radial anchors will be used to monitor the behavior of the anchors, consistent with the design. To monitor the behavior of the containment shell, strain gages are planned throughout the repair area between buttresses 3 and 4 and in different directions.

Note: Detailed layout and installation of radial strain gages is in the planning phase. The following discussion describes the proposed current plan:

Radial strain gages currently installed or approved to be installed per Attachment G02 EC 75220:

- Gages centered across the plane of maximum radial tension:
  - o One radial gage has been installed near the radial anchors below elevation 176. This gage is positioned with half in the original concrete and half in the new concrete at a location close to the center of the first concrete placement.
  - o One additional radial gage will be installed across the plane of maximum radial tension in Bay 23 Core Hole No. 72.
- Gages centered across other planes:
  - o One gage will be installed at the concrete interface near mid-height of and at the left side of the 24" excavation area in bay 34.
  - o One gage will be installed at the center of the original SGR opening in bay 34.

Additional radial strain gages to be installed per EC 75220 (not currently shown on Attachment G02)

- Gages centered across the plane of maximum radial tension:
  - o Two radial gages will be installed at the concrete interface and close to the radial anchors above elevation 230 in bay 34.
  - o Two radial gages will be installed at the concrete interface between elevation 176 and 230. These gages will be in the nominal ten inch deep excavation area close to buttresses 3 and 4.

The strain gages located in the radial direction will provide the strain related to the radial stresses in addition to the strain related to the Poisson's ratio effect (strain in the transversal direction to the direction of application of the load). Although this may not allow a direct reading of the radial tension, the strain gages will indicate whether a significant change has occurred.

All strain gages are being installed per EC 75220. A detailed description of the strain gages is located in attachment G02 of EC 75220.

(see response folder for attachment)

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/10/2010

Status: Closed

Date Closed: 10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number:	147		
Individual Contacted:	Paul Fagan	**Date Contacted:**	6/22/2010
Requestor/Inspector:	Louis Lake	**Category:**	Information Request

Request: As stated in Section 4.1 of Calculation S10-0021, the radial pressure of 28.22 psi (taken from CR3 calculation S09-0054) is used for the design of radial reinforcing bars. Calculation S09-0054 documents other cases where the radial pressure is calculated taking into account the tendon duct void resulting in a higher radial pressure. The use of 28.22 psi in calculation S10-0021, which is the minimum radial pressure calculated in S09-0054, is not supported by any technical justification and is in contrary to the results of Calculation S09-0054 which documents other conditions that closely represent the physical condition of the containment wall. Further, the root cause investigation of the CR3 containment delamination has identified that significant tensile stress concentrations exist around the hoop tendon sleeves under fully tensioned tendon condition and was a contributing factor in the delamination. Please provide justification for the use of 28.22 psi radial pressure.

References:

Response Assigned to:	Don Dyksterhouse	**Date Due to Inspector:**	

Response:

Misc Notes: Data input error…duplicate to request #141

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	
Status:	Closed	**Date Closed:**	7/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 148

Individual Contacted: Paul Fagan

Date Contacted: 6/22/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: CR3 has had an operating experience of two delaminations (containment dome and cylinder) where soft/friable aggregate resulting in low concrete tensile strength was considered a contributing factor. Calculation S10-0021 implements a theoretical method of determining the capacity of the new grouted radial bars without regard to the site specific concrete properties. Additionally, an increased compressive strength of 5800 psi (vx. 5000 psi per the CR3 current design basis) is used which further amplifies the theoretical capacity of the grouted anchor (it should be noted that the use of higher than 5000 psi compressive strength to increase concrete tensile capacity is the subject of another SIT question previously submitted). The maximum interaction ratio for the radial anchor design is 0.97 as shown on Page 17 of the subject calculation and this provides no tolerance for variability in the design input parameters used in this calculation. Considering the above and lack of site specific test data, the design methodology implementated in this calculation does not provide reasonable assurance of the capacity of the proposed anchoring system. Provide discussion relative to any planned testing of these anchors to ensure their capacity satisfies the design intent. Also, describe in detail the acceptance criteria and method of inspection for the grouting of these radial anchors.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Misc Notes: Data input error…duplicate to request #142

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 7/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 149

Individual Contacted: Paul Fagan

Date Contacted: 6/22/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: According to Section 4.4 of Calculation S10-0021, it is assumed that no shear stress will be resisted by the radial bars. The justification given in this section is not sufficient to disregard the effects of combined shear and tension stresses. Provide detail justification, in a quantitative form, for the exclusion of shear stresses from the design of new anchors when radial and plane shear stresses in the area of the new grouted anchors are expected due to the applicable design basis loading combinations.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Misc Notes: Data input error…duplicate to request #143

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 7/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 150

Individual Contacted: Paul Fagan

Date Contacted: 6/22/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: The new anchors are installed at the interface of old and new concrete. Provide information, in a quantitative form, relative to the shear and tension loads that the new anchors may be subjected to due to the loading conditions where bending of the shell is expected (e.g., accident thermal loading).

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Misc Notes: Data input error…duplicate to request #144

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 7/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 151

Individual Contacted: Paul Fagan

Date Contacted: 6/22/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: Calculation S10-0021 states that the new anchors will be installed where a nominal 14" of concrete has been removed. This implies that there is no plan to install anchors where a nominal of 24" concrete is removed. Provide information, in a quantitative form, the adequacy of the interface of old and new concrete (to act as a monolithic 42" thick wall) to resist the expected stresses (under all applicable design basis loading conditions, prestressing, seismic, thermal, pressure, etc.) without any anchors.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Misc Notes: Data input error…duplicate to request #145

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 7/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 152

Individual Contacted: Paul Fagan

Date Contacted: 6/22/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: Consistent with the dome delamination radial anchor repair (performed in 1976), provide a commitment that sufficient strain instrumentation will be installed for the radial anchors to ensure that the structure and the radial anchors are responding, during the post-repair test, as designed.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Misc Notes: Data input error…duplicate to request #146

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided:

Status: Closed

Date Closed: 7/29/2010

Request Number: 153

Individual Contacted: Paul Fagan

Date Contacted: 6/28/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: Attachment Z48R2 to EC 75220 describes vertical cracks observed in Bay 3-4 and Bay 5-6. It is our understanding that in Bay 3-4, horizontal and vertical cracks were identified after removal of the delaminated layer.

a. Provide information and quantitative evidence relative to the root cause of these vertical and horizontal cracks.
b. Provide quantitative evidence to refute or confirm the following:
i.Were these vertical and horizontal cracks developed due to the initial detensioning of 10 vertical and 17 hoop tendons, and concrete removal?
ii.Were these vertical and horizontal cracks developed due to further detensioning of a total of 64 vertical and 155 hoop tendons?
c. Provide the extent of condition, the examination method, and the repair method of horizontal cracks in Bay 3-4.
d. Since the horizontal cracks in Bay 3-4 were not visible at the surface and were only exposed by concrete removal to nominal 10” to 15”, how have you established that they do not exist elsewhere in Bay 3-4 or other bays (specifically, Bay 6-1 where 32 vertical tendons were detensioned) where concrete was not removed?
e. EC 75220 assumes that the vertical cracks below elevation 176 are due to shrinkage, creep or temperature effect. Considering the presence of horizontal and vertical cracks in Bay-3-4, provide the technical justification for this assumption in a quantitative form.
f. Provide the extent of condition of vertical cracks in all bays of the containment structure.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The question originates from a review of EC 75220 Attachment Z48R2 for cracking below elevation 176 in Bay 34. The question pertains to cracking in Bay 34 and to cracking in bays other than Bay 34. An apparent cause investigation of vertical and horizontal cracking was completed in NCR 395843. Details of the investigation are documented in the NCR 395843 Investigation Report. Both of these documents have been provided to the S.I.T. Discussion with the S.I.T. member that originated the question indicates that review of NCR 395843 and the Investigation Report will resolve the questions. For that reason, no additional response in addition to the two documents is being provided.

References:
NCR 395843
NCR 395843 Investigation Report

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/20/2010

Status: Closed **Date Closed:** 10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:37 AM

Request Number: 154

Individual Contacted: Paul Fagan

Date Contacted: 6/28/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: Attachment Z49R4 to EC 75220 discusses not repairing the vertical cracks below Elevation 176 and leaving them in-place. This attachment attempts to address the effects of these cracks on the design basis in a qualitative format. All FSAR sections referenced in this attachment discuss the design of rebar for load conditions, such as LOCA thermal loads, which have not occurred. These FSAR sections do not allow cracking in the containment structure as it is implied in this attachment. As it is stated in FSAR Section 5.2.3.3.1, “The concrete shell has been prestressed sufficiently to eliminate tensile stresses due to membrane forces from design loads.”

The design basis calculations are performed based on the assumption that the structure is uncracked and the prestressing force will be distributed based on a monolithic containment wall. Provide further information and justification, in a quantitative form, regarding the effects of leaving these vertical cracks on structural performance of the repaired structure as it relates to the design basis calculations performed for the CR-3 containment structure.

Supplemental Question:

Response to the question focused on the condition that exists in areas outside of Bay 34 because these areas were considered to be more critical. The previous response lacks a discussion regarding shear in Bay 34.

Second Supplemental Question:

Compare use of response spectrum analysis in the FSAR to the method used in Calculation S10-0058.

Third Supplemental Question:

Design calculations for reinforcing steel in Bay 34 include significant shear stresses for the 1.5P case. Discuss the effect on these shear stresses in light of vertical cracks in Bay 34.

Fourth Supplemental Question (01/05/11):

Attachment E “Testing Requirements” of the draft of EC75221R2 includes monitoring of vertical cracks in Panel N of Bay 23, Panel P of Bay 45, Panel N of Bay 56, and Panel J of Bay 61 to measure width of cracks at outside surface of the wall prior to retensioning, after retensioning, after pressure test, and at 1 year following completion of the repair activities.

In response to Question 154, the basis of your analytical calculations performed to demonstrate the acceptability of leaving these through-thickness vertical cracks “as-is” is that they will close after the containment is retensioned and will stay closed through the service life of the plant following return to service. We have the following questions pertaining to the method of monitoring and inspection described in the draft of EC75221R2 and its effectiveness to provide reasonable assurance that the vertical cracks will indeed close through the thickness of the containment wall.

1. Why does the monitoring plan of vertical cracks exclude Bay 12?

2. Your plans indicate that monitoring will be done in only one panel of Bays 23, 45, 56 and 61.

Monitoring of one panel may not sufficiently represent the stress variance in each bay as panels in close proximity of the buttress may have a different stress distribution than the panels in the middle of the bay. Provide justification relative to (1) the statistical validity of the sample size; and (2) stress variance representation of the sample size included in the vertical crack monitoring plan.

3. What physical evidence is being used, other than the crack width measurement at the outside surface of the containment, to confirm the closure of the vertical cracks after retensioning through the thickness of the wall?

4. Please confirm that the monitoring of the vertical cracks has been entered into your IWL program and will be implemented for the remaining service life of the plant.

Fifth Supplemental Question:

S.I.T. Question 154 originally requested quantitative justification regarding the effects of cracks in Bay 34 below Elevation 176 that were not repaired. Vertical hairline cracks have also been observed in the other bays that also are not repaired because initial prestress closes the cracks. The initial response addressed the effect of cracks on structural response and shear capacity for the overall structure with design basis end of life tendon forces. Supplemental responses have addressed in greater detail the shear capacity in Bay 34, methodology for the development of seismic loads, the potential impact to reinforcing steel design in Bay 34 and monitoring plans to confirm closure of cracks. In a conference call on January 31, 2011 between Progress Energy and NRC staff, the need for this most recent supplemental response #5 was discussed. The following items were requested to be addressed further by Progress Energy:

1. Justification for monitoring plans and scope was provided as requested in Supplement #4, but the lack of specific monitoring for closure of cracks in Bay 12 was not resolved sufficiently to justify continued omission. It was requested to be reconsidered.

2. A report has been provided to NRC addressing acoustic emissions monitoring. Early tendon tensioning sequences show evidence of crack closure. Additionally, strain readings and laser scans were discussed during the call showing response consistent with expectations. A sample of this data and the retensioning sequence was requested by the NRC to evaluate the interim response of the containment.

3. Monitoring for crack closure only once following containment repair (1 year surveillance) was not considered sufficient by the NRC for long-term assurance of crack closure. Increasing this frequency to include the vertical crack measurements in the containment 5 and 10 year surveillances was proposed by Progress Energy on the conference call and agreed to be documented further in a supplemental question response.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

The question directly pertains to hairline vertical cracks below elevation 176 in bay 34. However, the basis for leaving these cracks without repair is also applicable to the vertical cracks in bays other than Bay 34. Therefore, the response also includes references to the hairline vertical cracks in these bays.

An analysis in EC 75220 Attachment Z49R4 addresses the local effects of leaving the cracks that interface with vertical tendons. The existing concrete is cracked on the inboard side of the vertical tendons. New concrete is placed on the outboard side of the tendon. Based on inspection, a representative width assigned to each of the 16 locations is 0.003 inch. The EC attachment concludes that the cracks will close and have no meaningful effect on stresses that exist in the old or new concrete. Further analysis is provided below to address the global effects to the structure in a manner consistent with original design calculations. Finite element analysis techniques are also being use to evaluate this condition. The finite element analysis results will be included in Calculation S10-0058, “Evaluation of Vertical Cracks Outside the SGR Bay”.

Vertical cracks were observed in concrete where tendons were detensioned for repair of delaminated concrete in Bay 34. Tendons were detensioned in sets to avoid unbalanced conditions at the buttresses. Therefore cracks are conservatively assumed to exist for the full perimeter of the building. Vertically oriented surface cracking was observed in Bay 56, 61, 12 and 23. Cracks observed in Bay 34 are single vertical cracks that generally follow the vertical tendons. They occur at the same spacing as the vertical tendons. In other bays, ground penetrating radar was used to locate vertical tendons. The cracks also follow vertical tendons in other bays. In one location, an angled core was taken to confirm that the vertical crack exists behind the vertical tendon. Therefore, all vertical cracks are assumed to be through thickness cracks. The observed condition of the vertical cracks in other bays is very regular and uniform with an opening in the detensioned condition of less than 0.010 inch.

The cracks will close due to prestress. When tendons are tensioned and after elastic shortening and age-related losses, a compressive stress will be imposed across the crack face of at least (1435 kip)/A=1.646 ksi, where A=872 〖in〗^2 (Area of concrete and transformed steel effective with tendon tensioning.) The tendon force of 1435 kips is used because this is the new value for minimum required prestress force. This compressive stress is large and will close the cracks. This is also consistent with the conclusion in Calculation S10-0036, “Analysis of Vertical Crack Closure”.

Original Design:
The original design of the prestressing system credited concrete with 212 psi membrane tensile capacity per FSAR Section 5.2.3.3.1 for factored loads(3√(f`c)=212 psi, where f`c=5000 psi). In the original design calculations, hoop tendon losses due to elastic shortening and age related effects were included to ensure that end of life prestress was sufficient to keep concrete tensile stresses below 212 psi during the critical condition described as a winter LOCA (Loss of Coolant Accident). The FSAR also states that the concrete shell has been prestressed sufficiently to eliminate tensile stresses due to membrane forces from design loads. The worst design load described in Section 5.2.1.2 is pressure and temperature associated with the design basis LOCA (P + Ta). The factored condition controls for design of the prestress system because the effect of accident pressure is higher for the factored condition.

$$\sigma=PR/t$$

where: R=780 inch, Radius of containment liner
t=42 inch, Thickness of concrete
σ = 1532 psi for P = 1.5*55 psi
σ = 1021 psi for P = 55 psi

Therefore, the difference in concrete stress due to application of the load factor is 511 psi. Since 511 psi is greater than 212 psi, the factored case controls for design of the prestress steel.

Winter conditions have the effect of a slight reduction in prestress because concrete through thickness temperature is lower than when the tendons were initially tensioned. The LOCA pressurizes and heats the containment structure. Although the worst pressure and worst effect of temperature change do not occur at the same time, the prestress system was designed with that assumption. The LOCA results in internal pressure of 55 psi and a liner temperature of 281 degrees F. Since the critical design basis load combination for tendons uses a load factor of 1.5 for pressure, the prestress system was designed for the following limiting condition for concrete stresses:

$0.95D+F+1.5P+T_a \leq 212$ psi
where:
D = Dead load (has no impact for hoop tendons)
F = Prestress
P = Peak Pressure associated with the LOCA
Ta= Peak Liner temperature – Actually occurs after peak pressure

Prestress needed to satisfy this requirement resulted in a minimum end of life tendon force of 1252 kips for hoop tendons in the original design. Other variables such as tendon spacing and material properties were also selected in this design process but are now considered to be constants.

Increasing Prestress to Compensate for Tensile Capacity:
Detensioning hoop tendons on a global scale was required to repair concrete in Bay 34. All hoops between Elevation 170'-0 and 240'-0, two thirds of the hoops between Elevation 150'-0 and 170'-0 and 64 vertical tendons were detensioned. Because tendon forces will be reset to the original construction forces and the remaining life for the plant is less than 40 years, the end of life tendon forces will be higher than previously determined. Note: All 144 vertical tendons are being reset.

Concrete is currently known to have through thickness vertical cracks at regular intervals. No tensile capacity in the hoop direction can be realistically credited.

The 212 psi used in original design must be compensated for with increased prestress. This increase provides more compression in the concrete against which accident pressure will react. Not so obvious is the fact that the liner is also at a higher compressive stress. When the liner heats due to LOCA conditions, there is less of a change in liner stress to achieve the plastic condition. This creates a smaller strain and the effect to the concrete is also smaller.

A close approximation of tendon force needed to prevent hoop tension in the $1.5P+T_a$ case is to set the tendon force increase equal to the concrete tensile capacity that is being ignored (no longer credited).

$T-1252 \text{ kips}=0.212 \text{ ksi}*A_T$
$T=1437$ kips
where:
T = Required tendon force
1252 kips = Minimum required prestress from original design
0.212 ksi = Tensile capacity in original design
$A_T=872 \text{ in}^2$ Area of concrete and transformed steel for one hoop affected by tendon tensioning

Hoop forces greater than 1437 kips will ensure that no tensile demand is placed on the concrete.

Tensioning tendons to 70% GUTS (Guaranteed Ultimate Tensile Strength) consistent with the original construction will result in an average end of life force of 1461 kips per tendon based on conservative analysis of elastic shortening, shrinkage, creep and wire relaxation.

Because 1461 kips > 1437 kips, no tensile demand exists on the concrete for the critical factored load combination that includes 150% of accident pressure.

Shear Capacity:
Another possible effect of vertical cracks is the impact on shear capacity.

Design loads that require shear capacity on the vertical-radial plane are seismic and wind. Four factored load combinations that include seismic and wind are:

$1.25E+1.25P+T_A$

1.25W+1.25P+T_A
2.0E+1.0P+T_A
2.0E+1.0P+T_A
where:
E = 0.05g seismic loading
W = 100 year wind loading
P = Peak pressure associated with the LOCA
Ta= Peak liner temperature – actually occurs after peak pressure

The highest pressure of these cases is 125% the accident pressure. As shown above, no concrete tensile stress will exist due to 150% accident pressure, concrete compression associated with 25% of accident pressure will be available to ensure shear capacity during these factored load combinations. Concrete stress for 0.25P is:

0.25P/(1.5P)*(σ_c)=222 psi
where:
σc = Range of concrete stress cause by pressure of 0 psi to 1.5P alone.
σ_c=1.5P*R*S/A_T =1331psi
where:
P = 55 psi, Peak pressure associated with the LOCA
R = 780 in, Radius of containment liner
S = 19.3 in, Spacing of tendons
A_T=933 〖in〗^2, Area of concrete and transformed steel for one hoop affected by pressure loading

ACI 318-05 Section 11.7.4 “Shear Friction design method” allows use of μ = 1.0. Therefore, at least 222 psi is available for shear loading. Note: ACI 318-63 includes provisions for a similar situation where the allowable stress on this surface is 160 psi for service loads or 258 psi for factored loads (reference Chapter 25). This capacity is compared to (V*Q)/I stress for 1.25E loading for the structure above Elevation 150’-0. Elevation 150’-0 is the lowest elevation where tendons were detensioned. Inspections at this elevation and lower have revealed that vertical cracks do not exist.

Calculation S10-0033 conservatively determines shear flow stress for the SSE (2.0E) condition to be 139 psi. Shear stress for 1.25E is 139 psi*1.25/2.0=87 psi. Therefore, the shear capacity (conservatively assume 160 psi) is greater than the shear demand (87 psi) for this condition. Concrete shear capacity is acceptable for design basis loads considering the existence of vertical cracks.

Conclusion:
Leaving vertical cracks does not affect the structural performance of the repaired structure. Adequate shear capacity exists to preclude a change to the seismic response. Tendon forces have been increased such that cracks will not open for design loads.

Attachment:
None

References:
S10-0033, “Evaluation of Seismic and Deadweight in RB as a function of Elevation and Angle” (see request folder for reference)
S10-0036, “Analysis of Vertical Crack Closure” (see request folder for reference)
S10-0058, “Evaluation of Vertical Cracks Outside the SGR Bay”

Supplemental response (10/7/10):

Shear stresses for Bay 34 are addressed in Calculation S10-0030 Rev 1 (Reference 1). Shear capacity of the existing concrete is not credited in design of reinforcing steel for Bay 34. A comparison is made of worst case shear from all load combinations to shear capacity of the new concrete alone. Using this approach, the effect of hairline cracks in the existing concrete is not consequential. The lowest safety factor from this conservative check is 1.4. See page 55 of 1135 from Reference 1. (Attachment 1)

Additionally, Calculation S10-0012 Rev 1 (Reference 2) shows that membrane hoop stresses are compressive for cases that involve external loads. External loads such as seismic and wind are the critical considerations for shear. Reference 2, Tables 7-10, 19, 28, 37, 46, 55, 64 (Attachment 2) provide membrane hoop stress during these load cases. In all cases, the membrane hoop stress is compressive and therefore the cracks will be closed. Table 7-10 (1.25W Case) and Table 7-19 (1.25E Case) (Attachment 3) include the lowest hoop stress. For areas 5 ft wide by 7.6 ft high, the lowest hoop stresses are 79 psi for the 1.25W case and 102 psi for the 1.25E case. Comparison with the shear demand for these cases (Tables 7-16 and 25 for radial shear and Tables 7-18 and 27 for in-plane shear) over the same areas shows a safety factor of 1.3 and 1.5 for the 1.25W and 1.25E cases respectively (Attachments 3 and 4). These safety factors assume a coefficient of friction of 1.0 for the contact surfaces. Since 1971, ACI 318 recognizes that a value of 1.4 is applicable for this application. Use of a friction coefficient of 1.0 is conservative.

The actual shear stresses for these areas are determined by SRSS of stresses from the radial shear and in-plane shear. For these 2 cases, (1.25W and 1.25E) the maximum shear stresses are 130 psi and 143 psi which compare favorably to ACI 318-63 Section 2505 for the case where steel ties are used. Conservatively applying a phi factor (0.85) yields allowable shear stress of
0.85(1.9*160 psi)=258 psi per ACI 318-63. This results in a minimum safety factor of 1.8 which occurs for the 1.25E case.

Summary:
Shear in Bay 34 can be transferred by the new concrete with a minimum factor of safety of 1.4.

When shear stress in the existing concrete is compared with hoop compression, the minimum factor of safety is 1.3 against movement along the shear plane.

References: (see request folder for references)
Calculation S10-0030 Rev 1, “Reinforcement Design for Delaminated Containment Wall”.
Calculation S10-0012 Rev 1, “Stresses Around the SGR Opening due to Design Basis Load Cases”.

Attachments: (see request folder for attachments)
Attachment 1 - S10-0030 R1, Attachment A, Page 55
Attachment 2 - Membrane Hoop Stresses for Cases with External Load
Attachment 3 - Radial Shear Stresses for Cases with External Load
Attachment 4 - In-Plane Shear Stresses for Cases with External Load

Second Supplemental Response (11/16/10):

CR3 is a rock site with minimum seismic demand. The OBE is 0.05g and the SSE is 0.10g. The OBE is defined and the SSE is taken as 2 times the OBE. Damping for the Reactor Building shell is described as 2% for the OBE and no modification is made for SSE (conservative). Vertical response is defined as 2/3 of the horizontal. Absolute summation is used for modal combination. The FSAR discusses 2 distinct analyses of the Reactor Building shell. In one case, it was analyzed as a shell of revolution. The model is described in Figure 5-21. The second analysis uses a stick model as described in Figure 5-28. Modes from the shell of revolution analysis are not provided. For the stick model, the first mode is 4.4 Hz. The highest mode reported was 98.4 Hz. The FSAR concludes that fixed base analysis is reasonable and conservative.

Seismic analysis in Calculation S10-0058 is also a shell of revolution analysis with the first mode at 4.2 Hz. Input motion is the 2% damped spectrum. Absolute sum combination of modes was performed with modes up to 95 Hz. This model also uses a fixed base.

Third Supplemental Response (11/16/10):

In general, the hairline vertical cracks observed coincident with some of the vertical tendon ducts were evaluated to ensure that compressive forces across the crack from post tensioning are great enough to prevent any impact from shear stresses.

The general approach for these cracks in Bay 34 was to excavate the crack and replace with fresh concrete. Some of the hairline vertical cracks also extend into the heavily reinforced parts of Bay 34. These hairline cracks were not excavated but new concrete was placed over these areas. Additionally, analysis was performed to ensure that the cracks would close under post tensioning. As a simple remedy for the potential effect of cracks in Bay 34, analysis is included in Calculation S10-0030 to ensure that new concrete (15 inch thick) was capable of transmitting shear from design load cases. However, other effects of cracking on performance of the structure are:

1. Potential impact to the response characteristics, or
2. Potential impact to reinforcing steel located in the old concrete.

Item 1:
The response characteristics of the structure determine the magnitude of dynamic loads. Forcing function responses can be amplified or damped based on structural configuration and material properties/conditions. Primarily, the seismic response of the structure is driven by the stiffness and material damping. Other similar effects can occur for wind design of the structure but not for short extremely stiff buildings for a nuclear plant.

Cracking can affect both parameters for seismic design. If cracks in concrete are allowed to open or displace during a seismic event, the stiffness is reduced and in most cases, this results in higher amplification. Damping is also increased and therefore, the response of the structure would be reduced.

However, these considerations do not apply for the 1.5P case. Seismic loading is not postulated concurrent with the load case where the design basis accident pressure is increased to 150%. The 1.5P case only includes accident pressure and the associated thermal conditions. Therefore, even though this load case includes higher shear stresses than load cases discussed in first supplement response to this question, there are no consequences to the response of the structure.

Item 2:
These hairline cracks do not reduce the capacity of reinforcing steel. The steel design for factored load cases uses Ultimate Strength Design (USD) where cracking of concrete is expected. The USD method assumes the nominal strength of a section is reached when the strain in the extreme compression fiber is equal to the crushing strain (ϵ=0.003) of the concrete. Obviously, the vertical cracks have no effect on the ability of the concrete to resist compressive forces and can be ignored. When crushing occurs, the strain in the tension reinforcement may be either larger or smaller than the yield strain dependent on the relative proportion of steel to concrete. Since the concrete in the tension zone is assumed to be fully cracked in USD and ignored in the evaluation of the nominal strength of the cross-section, the presence of any vertical cracks is of no concern.

Fourth Supplemental Response (01/07/11):

1. Why does the monitoring plan of vertical cracks exclude Bay 12?

Response: (See the response to question 2.)

2. Your plans indicate that monitoring will be done in only one panel of Bays 23, 45, 56 and 61. Monitoring of one panel may not sufficiently represent the stress variance in each bay as panels in close proximity of the buttress may have a different stress distribution than the panels in the middle of the bay. Provide justification relative to (1) the statistical validity of the sample size; and (2) stress variance representation of the sample size included in the vertical crack monitoring plan.

Response: Crack Monitoring Sample Selection
Design of the containment shell is based on a uniform cylinder (shell of revolution). Stresses were taken as uniformly distributed around the perimeter in the original design. There are no penetrations or other discontinuities (except for repairs in Bay 34) in the portion of the structure where vertical hairline cracks have been observed. This condition was created by a single event. Detensioning tendons after 33 years of creep strain resulted in compression in the liner and embedded steel and tension in the concrete. The tension was relieved by uniform cracking around the entire circumference. Additionally, the cracks are very similar in width. The measurement variation is very consistent with all cracks being within a narrow band around 0.005 inches. Once tendons are tensioned, concrete prestress is about 1700 psi around the entire circumference. Since tendons span 120 degrees but start every 60 degrees, frictional losses in force that occur between the end points are offset by the next adjacent tendon. The average force of every pair of tendons separated by 12.75 inches is therefore considered constant.

Due to uniform design, loading and prestress, the population for this inspection has been considered to be one. Closure of cracks at any single location around the perimeter indicates that sufficient prestress exists at all locations around the perimeter to ensure closure everywhere. However, to be more conservative, 3 equally spaced locations were initially selected (Bay 23, 45 and 61). Considering that 5 bays were available, this was considered to be a very large fraction. The sample has since been expanded and diversified. Four samples are now being used to observe 2 mid-panel areas and 2 areas that are adjacent to buttresses. There are 5 mid-panel areas and 10 areas adjacent to buttresses. This can be considered to be a population of 15. A sample size of 3 results from applying normal inspection criteria in MIL-STD-105E. Therefore, the selected sample size of 4 is considered conservative.

Additionally, 2 of the locations selected were those where the crack was verified to be more than just a surface indication. The location in bay 5-6, near core bore 139, was based on its location near the center of the bay and the core bore was performed directly over the crack to characterize the extent of the crack. The location in bay 6-1, near core bore 142, was based on its location is near the buttress and the core bore was performed at an angle that intersected the crack to characterize the extent of the crack.

3.What physical evidence is being used, other than the crack width measurement at the outside surface of the containment, to confirm the closure of the vertical cracks after retensioning through the thickness of the wall?

Response: There are two objectives for this investigation. Protection of the embedded steel is accomplished by limiting crack widths utilizing guidance from ACI 224R-01. Final measurements are needed to ensure the guidance in ACI 224R-01 is met. Contact is the goal for structural analysis. Structural response characteristics are maintained by achieving contact. Calculation S10-0058 includes further evaluation of lateral resistance capability where surfaces are in contact. This objective is confirmed by further analysis with the final measurements and other available data.

As discussed in #2, design of the cylinder is uniform and the observed surface condition is also uniform. Crack widths are consistent and the distribution is uniform. (see request folder for next sentence/figures) - Geometry requires that radial movement (□r) that opens hairline cracks on the exterior surface results in a crack width of □•□r regardless of the radial distance where the measurement is taken (where □ is the azimuthal difference between the crack locations). Therefore, confirmation of crack closure on the outside surface is representative and monitoring for crack closure in the inside surface is not required. In addition, the acoustic emissions monitoring data will be utilized in the assessment that the vertical cracks are responding as expected.

4. Please confirm that the monitoring of the vertical cracks has been entered into your IWL program and will be implemented for the remaining service life of the plant.

Response: An Owner Elected – Augmented inspection to monitor a sample of the vertical cracking has been added to the CR3 Containment IWE/IWL Inspection Program. This inspection is to measure the widths of the selected vertical crack sample following the tendon retensioning, after the repair pressure tests, and during the 1 year required containment repair concrete examination at a minimum. After completion of these additional inspections, the results will be assessed for consideration of continuation.

Note: The ASME Section XI, Subsection IWE examination requirements for concrete surfaces are not being altered or changed due to this augmented inspection. The concrete surface in the areas of these vertical cracks will continue to be examined per the requirements established in the CR3 Containment IWE/IWL Program for evidence of conditions indicative of damage or degradation such as described in ACI 201.1 and ACI 349.3.

References:

MIL-STD-105E - Military Standard - Sampling Procedures and Tables for Inspection by Attributes, May 10, 1989
ACI 224R-01, Control of Cracking in Concrete Structures, May 16, 2001

Fifth Supplemental Response (02/14/11):

Item 1: The previous response #4 described the uniform nature of loads, uniformity of cracks and uniformity of prestress forces that close cracks. Justification was provided for the scope of four monitoring locations as a representative sampling of this homogeneous population. However, since monitoring of Bay 12 was the only bay outside of Bay 34 not included, it was requested to be considered.

Monitoring of Bay 12 will not entail a significant consequence regarding manpower, dose, personnel safety, documentation or reporting and therefore was included in Rev. 8 of Engineering Change (EC) 75221. A location was selected in Bay 12, Panel AD and is being added to the IWE/IWL program. Measurements of this new location were taken on February 2. It should be noted that this measurement was unlike the planned baseline measurements taken in the other four locations immediately before retensioning commenced. At the time of this measurement, Pass 4 of the 11 retensioning passes was in progress. As a result, this first measurement was significantly below the previously observed 5 mils value at this location. That last measurement was taken in the Fall of 2010 during detailed mapping of all vertical crack indications in bays outside Bay 34.

Item 2: Radial strain gauges throughout containment have been functioning since the beginning of retensioning and are being continuously monitored as part of retensioning oversight along with acoustic emissions monitors. Radial strain is heavily influenced by hoop strain. In order to register any significant radial strain, closure of any vertical cracks must complete. Attached is a plot of radial strain for three gauges (CRT 71A, B and C) since the beginning of retensioning. These gauges are located at Elevation 205 and Azimuth 30 degrees (Bay 12) and were selected as a representative location to illustrate the retensioning effects. Other radial strain gages have exhibited similar trends. The A, B and C designations relate to 7, 10 and 13 inch depths, respectively. The short term effect of thermal cycling can be observed in the plot. As expected, thermal effects are more prominent for the "A" gauge than the others. Readings were not re-baselined at the beginning of retensioning. This prevents the plots from overlapping.

A plot of laser scan data is also provided as an attachment to this supplemental response. Laser scan measurements of internal containment wall displacements have been taken at the end of each retensioning pass and at several pre-planned points within Passes 1 and 3. The average radial displacement from retensioning through Pass 5 is compared to a similar point during detensioning. Retensioning radial movements were adjusted to match at points of equal tendon force. In the graph, the entire right side was increased until the radial movement at the end of Retensioning Pass 5 equals a point on the left side with equal tendon force. This results in a vertical line along the ordinate of 0.13 inch which would be expected due to creep recovery. Estimated creep recovery using the Yue and Taerwe approach from ACI 209.2R-08 is about 30% of the elastic strain recovered due to detensioning or 0.11 inch. The shape of the radial displacement curve is similar for detensioning versus retensioning and the shape of the structure is being restored as predicted.

The approved retensioning sequence is also attached to this supplemental response. This sequence was engineered to restore the containment from the scope of detensioning and repairs, while averting any potential for delamination damage. It utilizes partial tensioning for horizontal hoop tendons. This entails pulling each tendon to 50% of its specified lock-off value in Passes 1 through 4 and then fully tensioning it to 100% in subsequent Passes 8 through 11.

The monitoring of containment response during retensioning is providing the expected interim responses that are consistent with the modeling of the retensioning sequence.

Item 3: Future monitoring plans are included in EC 75221 Attachment E. As discussed during the 1/31/11 conference call, initial crack closure was observed with very slight tendon forces during the initial phases of retensioning. The force required to maintain closure is approximately the same as the force to initiate closure and is only a fraction of the expected end of life tendon forces. Therefore, significantly more than sufficient prestress will be available to maintain closure even at the end of life. Planned crack measurements during the 1 year surveillance following containment repair is confirmatory only. Regardless, as discussed on the 1/31/11 phone conference, two additional surveillances are being added to the IWE/IWL program plan. Specifically, the five selected vertical crack locations will be measured following start-up as follows:

•At 1 year (±3 months) following completion of the repair/replacement activity.
•At the 40th (2016 ±1 yr) year Tendon Surveillance
•At the 45th (2021 ±1 yr) year Tendon Surveillance

In summary, measurements will be performed at the 1, 5 and 10 year intervals as augmented IWL examinations. As with any surveillance activity, an engineering evaluation will be required to assess any unexpected results. In addition, after three confirmatory measurements following startup, an assessment will determine whether any trend has been established that would warrant further confirmatory measurements. As discussed on the telephone conference, regardless of the status of the augmented vertical crack measurements, all of the vertical crack locations will continue to be monitored per the normal IWL containment monitoring program.

(see request folder for attachments)

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/20/2010

Status: Open

Date Closed:

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

Request Number: 155

Individual Contacted: Paul Fagan

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: (Associated with Calculation S10-0036 rev. 0) Provide further information relative to basis for the strain value of 20 E -6 for the shrinkage of new concrete.

Follow-up Question:
Provide the value for shrinkage of the new concrete.

References:

Response Assigned to: Don Dykstерhouse

Date Due to Inspector:

Response:

The effect of shrinkage was evaluated in S10-0036, Analysis of Vertical Crack Closure, to determine if expected shrinkage is beneficial to crack closure and to the distribution of stresses through the section. A value of 20μ in/in was conservatively used because it is less than the shrinkage value determined by test for the new concrete. The results indicate that even with a small value for shrinkage, the stresses decreased. Using a higher shrinkage value would result in a further decrease of stresses.

Follow-up response (11/1/10):

EC 75220 Attachment Z14 includes test results of the concrete mix used in Bay 34. Pages from the EC are included. The autogenous (shrinkage due to the hydration/chemical process) shrinkage value is 114 μ in/in and the total shrinkage (shrinkage in un-sealed specimens) is 358 μ in/in for 91 days. Adjustments for humidity, temperature, and period of curing or volume to surface ratio on drying shrinkage have not been made. Even with those adjustments, 20 μ in/in is still considered to be a very conservative value for the study.

Attachment: (see request folder for attachment)

1. Pages from “EC 75220 Rev 21 Attachment Z14R21”

Misc Notes:

Response By: Don Dykstеrhouse

Reviewed By:

Date Response Provided: 9/29/2010

Status: Closed

Date Closed: 11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 156

Individual Contacted: Paul Fagan **Date Contacted:** 7/13/2010

Requestor/Inspector: Louis Lake **Category:** Information Request

Request: (Associated with Calculation S10-0036 rev. 0) Provide further information relative to the outcome of this the calculation if the liner plate is included in the model, consistent with the 3-D ANSYS FEA of the containment structure.

Follow-up question:

The follow-up question has to do with the tendon force used in Calculation S10-0036. It's not obvious that a conservative value was used. Including the liner in the model will affect the amount of prestress as will further consideration of actual creep and shrinkage. Provide further information to quantify these effects.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

The original response is completely replaced with the following (11/3/10):

Inclusion of the liner plate in the model would reduce the prestress available to close the crack. The liner stress at return to service is about 12 ksi because the concrete stress is about 1700 psi and the modular ratio is about 7.25. Therefore, the liner reduces the force available to the concrete by 87 kips (tendon spacing is 19.3 in and the liner is 3/8 in thick). Therefore the predicted tendon force at return to service would need to be reduced by 87 kips to properly account for not including the liner in the model. The actual reduction was only 61 kips. This difference is compensated for by actual lock-off values. Return to service predictions assume lock-off values of 70% GUTS. The minimum lock-off value is 70% GUTS. Actual tendon lock-off forces range between 70% and 74% GUTS. One half of the range equals 45 kips. Therefore, accounting for the liner plate (87 kip reduction) and actual tendon lock-off forces (45 kip increase) results in 1561-87+45=1519 kips. This tendon force is higher than the value used for the study and is therefore acceptable for return to service values.

Tendon losses occur over time that could eventually reduce the tendon force to a value less than the value used in this study. The minimum design tendon force for the "retensioned" group of hoop tendons is 1435 kips. This value is used in calculations for restoration of the containment and is being included in the tendon surveillance program as the design requirement for the end of extended license (60 years). The predicted hoop tendon force is 1461 kips for "retensioned" tendons. Tendon losses are also accompanied by beneficial effects of time. A difference in prestress between the new and old concrete will cause the higher stressed portion to creep at a higher rate than the adjacent concrete. The effect of a difference in prestress of as small as 100 psi would close a 0.0007 inch crack (creep coefficient = 1.1, E = 6 Msi, crack spacing = 36 inch). Therefore, end of license conditions are not considered to be controlling because stresses will balance over time.

Lastly, Calculation S10-0036 was needed primarily to determine if crack repair was required for the 24 inch excavation area. The conclusion was to perform repair. The only cracks not repaired are cracks that are about 0.005 inch wide and interface with a vertical duct at their end point. Calculation S10-0036 concludes that all such cracks (0.003 – 0.015 inch) will close for the 15 inch excavation area because of the interface with the vertical duct. Considering that a 0.015 inch crack will close with 1500 kips of tendon force, a 0.005 inch crack (one third the width) will also close for slightly less tendon force.

This calculation is not used for any other purpose. The initial response to S.I.T. Question 154 references Calculation S10-0036 for completeness. That response focuses on vertical cracks being left in other bays because they are considered to be controlling over cracks left in Bay 34. A reference is made to Calculation S10-0036 as a similar application of prestress to close cracks. The response to S.I.T. Question 154 has since been supplemented to directly address the vertical cracks and shear stresses in Bay 34. The original response, although valid is no longer germane.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/29/2010

Status: Closed

Date Closed: 11/10/2010

Request Number: 157

Individual Contacted: Paul Fagan

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: (Associated with Calculation S10-0036 rev. 0) Page 9 of the calculation states, that hoop tendon force at RTS is used. Provide further info relative to the outcome of the this calc when the EOL hoop tendon force is used.

Follow-up question:

Provide further information relative to reduction of tendon forces over time and the effect on crack closure.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The most critical condition has been modeled using return to service tendon forces. The premise applied in the calculation was that if cracks close, including the crack tip, due to prestress without causing a stress gradient, there is no structural impact. Increased prestress (as would occur for higher return to service tendon forces) allows the crack to close but cause a higher stress gradient. The crack tips close because the only cracks not being repaired in Bay 34 are those that intersect with tendon ducts or similar embedded objects. As discussed in Section 7.0 of the calculation, all cracks close that intersect with a tendon duct.

In summary, if end of life hoop tendon force was used, the stress gradient would be acceptable for even larger cracks. The cracks, including the crack tip, would still close.

Follow-up response (11/3/10):

Further discussion is provided in the supplemental response for S.I.T. Question 156.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 8/24/2010

Status: Closed

Date Closed: 11/10/2010

Request Number: 158

Individual Contacted: Paul Fagan

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: (Associated with Calculation S10-0036 rev. 0) Vertical tendon prestress force which has not been considered in the analysis would add an additional component of tension perpendicular to the vertical crack due to poisson ratio affects that would effect and inhibit the potential for crack closure. Provide justification for excluding this effect in the analysis.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The current plan for tensioning tendons includes the following sequence:

1. Detensioned state - Hoop tendons between the basemat (Elevation 93'-0) and about Elevation 150'0 remain tensioned. Most tendons between about Elevation 150'-0 and Elevation 240'-0 have been detensioned. The top three sets above Elevation 240'-0 remain tensioned. Eighty of the 144 vertical tendons remain tensioned.
2. Tension 64 vertical tendons to 70% GUTS.
3. Reset 80 vertical from their current setting to 70% GUTS.
4. Tension 155 hoop tendons to 70% GUTS.

Note: Step 3 may be performed before, during or after Step 4.

POISSON EFFECT:
Consistent with original construction, vertical tendons are being tensioned before hoop tendons. Assuming no hoop tendons were tensioned, tensioning vertical tendons causes hoop tension due to Poisson effect. Specifically, the difference in Poisson's ratios between the concrete and steel causes concrete tension. This example represents the original construction condition after all vertical tendons are tensioned and before any hoop tendons are tensioned. Tensioning the vertical tendons first is the worst case for tension on the concrete. This is because the hoop tendons create constraint. If hoop tendons were tensioned prior to the vertical tendons, any additional vertical prestress would cause an increase in hoop tendon tension and associated concrete compression.

Vertical Prestress:
Tensioning vertical tendons causes stress on the cross sectional area of the concrete plus the transformed area of the liner.

$\sigma_\varphi = P/A_T = 1.045$ ksi

where: $P = 1630\ \text{kips} \cdot N = 234720$ kip, Tendon Force
$N = 144$, Number of Tendons
$A_T = \pi \cdot D_2 \cdot t_2 + (\pi \cdot D)_1 \cdot t_1 \cdot n = 224{,}703\ (\text{in})^2$, Transformed Area
$D_1 = 2 \cdot 780\ \text{in} = 1560$ in, Diameter of Liner
$D_2 = D_1 + t_2 = 1602$ in, Middle Diameter of the Wall
$t_1 = 0.375$ in, Liner Thickness
$t_2 = 42$ in, Concrete Thickness
$n = 29/4 = 7.25$, Modular Ratio

Vertical strain:

Vertical displacement (shortening) of the structure due to prestress in the vertical direction will occur at the following rate.

$\epsilon_\varphi = \sigma_\varphi / E_c = 261\ \mu$ in/in

where: $E_c = 4.0 * 10^6$ psi, Young's Modulus of Concrete

Concrete Hoop Strain from Vertical Prestress:
The Poisson effect causes growth of the concrete and liner in the hoop direction. The two materials have different Poisson's ratios so a force is created. Two postulated strains are calculated as:

$\epsilon_{\theta c} = \epsilon_\varphi * v_c = 52\ \mu$ in/in
$\epsilon_{\theta s} = \epsilon_\varphi * v_s = 78\ \mu$ in/in

where: $v_c = 0.2$, Poisson's Ratio for Concrete
$v_s = 0.3$, Poisson's Ratio for Steel

Additional Liner Stress:
The difference in strain cannot occur and a compressive stress is created in the steel approximately equal to the difference in predicted strain times Young's modulus. Note: The stiffness of the concrete is about 15 times that of the steel liner.

$\sigma_s = (\epsilon_{\theta s} - \epsilon_{\theta c}) * E_s = 754$ psi

where: $E_s = 29 * 10^6$ psi, Young's Modulus of Steel

Additional Concrete Stress:
The compressive stress/force in the steel causes an equal opposite force in the concrete. This results in a slight tensile stress in the concrete.

$F = \sigma_s * t_1 = 283$ lb/in
$\sigma_c = F / t_2 = 7$ psi

This is a very small contribution compared to prestress of about 1700 psi in the hoop direction.

OTHER EFFECTS OF VERTICAL PRESTRESS:
An additional effect of vertical prestress in the hoop direction is shown in Calculation S10-0003, "Conduit Local Stress Analysis", Figure 7-2, Attachment A, page 13 of 15. Although this calculation does not represent any specific condition that exists during the evolution of containment restoration, it does provide an indication of the local stress effect due to vertical tendon tensioning. Vertical tendon force of 1474 kips results in high local tension at the 12:00 and 6:00 position of the hoop tendons. This stress is in the radial direction. The crack condition evaluated in Calculation S10-0036 is a vertical-radial crack inboard of the vertical tendon. The crack will not be influenced by stress concentration effects on the top and bottom of the hoop tendons because of the distance from the stress concentration to the vertical crack location on the inside of the vertical tendon. Additionally, the stress is not orthogonal to the crack surface.

In summary, the effect of vertical prestress was not included in S10-0036 because the effect is very small and is not expected to influence crack closure.

References:
S10-0003 Conduit Local Stress Analysis

Misc Notes:

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	9/23/2010
Status:	Closed	**Date Closed:**	12/17/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 159

Individual Contacted: Paul Fagan

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: (Associated with Calculation S10-0036 rev. 0) This calc assumes that epoxy will be injected into cracks. Provide confirmation that this is consistent with the repair plan and appropriate material properties has been included in the analysis.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Although injection of epoxy into the cracks was evaluated in calculation S10-0036, epoxy was not used to repair the vertical cracks in bay 34.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/17/2010

Status: Closed

Date Closed: 10/29/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 160

Individual Contacted: Paul Fagan

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Information Request

Request: (Associated with Calculation S10-0036 rev. 0) The analytical model shown in fig 3-2 assumes only 14” excavation of the crack with 4” crack length remaining on the liner plate side. The results of the analysis for this condition as stated in section 7.0 of this calculation show that 10 mil cracks will not close, a 3 mil crack closed within 1.25” and a 5 mil crack closed within 2.5”. Provide further information, in a quantitative form relative to the effects of these open cracks on the design basis calculation and on the integrity and leak tightness of the liner plate.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Although calculation S10-0036, Analysis of Vertical Crack Closure, evaluated leaving a 4” crack in the vertical direction near the liner, the repair implemented in EC 75220 excavates the entire crack.

The option to leave a 4” ligament was not implemented. Therefore, the repaired vertical crack will completely close and the leak tightness of the liner is not impacted.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/23/2010

Status: Closed

Date Closed: 10/19/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number:	161		
Individual Contacted:	Charles Williams	**Date Contacted:**	7/13/2010
Requestor/Inspector:	Louis Lake	**Category:**	Question

Request:

As discussed in the PII report, the analysis of the Crystal River containment indicates that the delamination ultimately depends on complex local conditions that are impacted by the response of the building as a whole. As a result, PII created a series of computer codes to simulate the entire building and executed detailed calculations for the particular area of interest. The global displacements, stress state, creep law, and the concrete damage plasticity parameters are all implemented into the sub-models. A custom designed modeling technology was developed to allow the use of creep, concrete damaged plasticity, sub-modeling, and stress state mapping, all at the same time. This technology remains the proprietary property of Performance Improvement International.

However, the following items are not clear to the reviewer:

(1) What is the damage approach used in modeling the crack initiation/crack growth for evaluating the delamination event (e.g., smeared crack or discrete crack method)?
(2) What are the internal variables and their values used for the specific constitutive and damage models chosen?
(3) Is the shear failure mode considered in the PII modeling effort? An important feature of the concrete cracking model is that, whereas crack initiation is primarily based on Mode I fracture, post-cracked behavior of concrete will include Mode II (or Mode III), as well as Mode I, especially in a biaxial prestressed concrete region. The Mode II shear model is based on the common observation that the shear behavior is dependent on the amount of crack opening.
(4) What is the basis of using predefined delamination regions during the FEM analyses (if my interpretation of PII report was correct), instead of letting/using the FEM analysis results to predict/determine the delamination/cracking sites? What cohesion bonding was assigned at these interface regions, if any, during the crack initiation stage of the finite element evaluation? Generally, cracking dominates the material behavior when the state of stress is predominantly tensile. Typically, the FEM model uses a “crack detection” plasticity surface in stress space to determine when cracking takes place and the orientation of the cracking. Damaged elasticity is then used to describe the post-failure behavior of the concrete with open cracks. Currently there are two common ways of dealing with cracking: smeared cracking and discrete cracking,
(5) The PII report has discussed the mesh size effect on the outcome of the predicted stress level in the global modeling. In order to cope with the mesh size effect, a smaller fracture energy and the reduced tensile failure stress level were used for damage level prediction in the PII global FEM analyses. The resulting predicted delamination sites based on these assumed values can be troublesome, due to artificially assigned damage criteria that are inconsistent with the concrete damage characteristics/properties. The reduced damage intensity criteria used to describe concrete properties seems merely to back fit the FEM results to the observed CR3 delamination data. Therefore, I would recommend that PII retain the genuine material failure properties and use the following steps to overcome the mesh dependence issues:

(i) First, perform a global model (relatively coarse mesh) evaluation to obtain the controlled boundary conditions (stress & displacement fields) for the subsequent submodel evaluation, with the fine mesh utilized to model the localized structure components. This may require more than one step of submodeling to optimize the mesh refining process,
(ii) Based on the submodel's results, integrate the damage regions into a global model with new crack surfaces or delamination zones (which in term form the new B.C.s); and then carry out the second round of FEM analyses that repeat (i)&(ii)'s action,
(iii) Upon the completion of the second round FEM study, compare the convergence/consistance

(i.e., any new damage zones/cracks or delamination site) of the new round of FEM results to the earlier results. If the concrete structure damage profile/topology of FEM simulation is stabilized, then, stop the FEM simulation. Otherwise, continue another round of iteration of FEM analyses, i.e., repeat (i) & (ii).

In another note, normally the coarse aggregate serves as a crack-arrestor in the concrete due to its higher strength and density. The presence of soft limestone aggregates introduces weakened locations where cracks may originate, or where existing cracks may propagate. Nevertheless, from an energy absorption viewpoint it also can effectively slow down/ or arrest/reduce the fast crack growth rate (this resembles a void phase being an effectively crack arrester, as shown in typical fracture mechanics testing). The CR3 concrete had up to 50% of these weaker particles, which would have a significant influence on the prestressed concrete cracking behavior.

Mode II/III shear fracturing is not generally available in commercial FE codes. Improvements in this area would lead to greater confidence in analytical results. This type of failure can also be a factor for prestressed concrete regions. There is a lack of data in respect to residual tensile strength, fracture energy, the influence of the load path (loading/heating/cooling sequence) and multi-axial effects considering the response of aged prestressed concrete. In prestressed concrete, the residual strength is of particular interest. Especially, during fault conditions, the inner vessel surfaces heat up and this induces compression of the inner surfaces and tension on the outer cooler surfaces (which may lead to cracking). Due to the development of irrecoverable creep during the heating phase, residual tensile stresses and cracking may occur in the previously compressive region of the prestressed concrete during subsequent cooldown, or in the detension scenario of the CR3 case. The axial tendons of CR3 containment are not located at the containment wall geometry center, but are closer to the outer wall surface. This has potential to substantially increase compressive stress at the outer wall region due to vertical tendon tensioning induced bending moment. Thus, detensioning of the vertical tendons alone is equivalent to inducing significant tensile stresses into the outer wall region.

Background:

It is generally accepted that concrete exhibits two primary modes of behavior:
(1) a brittle mode in which microcracks coalesce to form discrete macrocracks representing regions of highly localized deformation, and
(2) a ductile mode where microcracks develop more or less uniformly throughout the material, leading to nonlocalized deformation.

Moreover, the brittle behavior is associated with cleavage, shear and mixed mode fracture mechanisms that are observed under tension and tension-compression states of stress that almost always involves softening of the material. The ductile behavior is associated with distributed microcracking mechanisms that are primarily observed under compression states of stress. It almost always involves hardening of the material, although subsequent softening is possible at low confining pressures.

The cracking model described by PII apparently only considers the brittle aspects of concrete behavior. However, with the soft aggregate and prestressed concrete, it would be prudent to also examine the other aspect of fracture behavior.

In general, the quasi-brittle behavior of concrete in tension can lead to cracks which can potentially propagate through the body of the material. This is one of the key aspects of concrete behavior that requires simulation of the following features:

a)crack initiation,
b)crack layout: the layout, distribution and direction of cracks,

c)crack growth: how cracks grow and whether cracks merge or develop into secondary or tertiary cracks,
d)crack width,
e)temperature dependency: on crack initiation thresholds, and the associated thermal cycling,
f)loading rate dependency: stiffness and tensile strength can be loading rate dependent, and
g)load path dependency: in particular arising from crack induced anisotropy.

According to the Hillerborg's energy model, stress levels lower than the concrete tensile strength may result in partial damage and redistribution of local stress. As such, tensile strength or principal stress alone is misleading relative to determining the likelihood of crack propagation.

Cracking dominates the material behavior when the state of stress is predominantly tensile. Typically, the FEM model uses a "crack detection" plasticity surface in stress space to determine when cracking takes place and the orientation of the cracking. Damaged elasticity is then used to describe the post failure behavior of the concrete with open cracks. Currently there are two common ways of dealing with cracking: namely, smeared cracking and discrete cracking, as described below.

Smeared Cracking
The smeared cracking approach treats cracking as a constitutive material behavior (as opposed to a geometrical discontinuity in the case of discrete cracking). The concrete is represented with standard continuum elements and the material law for these elements allows cracking to occur. In this way, prior to the formation of any cracks, the concrete is considered to be an isotropic material. The key issue of using smeared cracking is "dependency on mesh density." Normally, the mesh dependence is resolved by adjusting the form of the tension softening diagram for each element, such that the work done in fully opening a crack is equivalent to the fracture energy of the concrete involved.

Discrete Cracking
With the discrete cracking approach, interface elements are introduced into the mesh on the path which the crack might propagate along. These interface elements are given appropriate material characteristics, for example a limited tensile strength, a frictional shear capacity, and an appropriate tensile stress-displacement relationship. A major drawback of this approach is that prior knowledge is required of the potential location of cracks and the direction in which they might grow. A common work-around for this is to perform an initial smeared cracking analysis before a discrete cracking analysis to obtain information about the likely location and direction of crack growth.

The discrete cracking method is often a good technique to use for unreinforced or lightly reinforced concrete which tends to form a small number of comparatively large cracks. It is not so suitable for heavily reinforced concrete, which tends to experience a large number of smaller cracks. Discrete cracking involves considerably more time to define the model and requires prior knowledge of crack locations and directions.

Characterization of Strength Envelope: Typically Drucker-Prager or Mohr Coulomb yield criterion can be used to define the failure surface. Mohr Coulomb has sharp corners which can give rise to numerical problems. Some programs modify Mohr Coulomb by rounding off the corners. Drucker-Prager generally includes a tensile cut-off: this also introduces discontinuities.

In another note, the accumulated low level damage cannot be effectively calibrated with conventional compressive or tensile experiments. However, it can be accurately calibrated through a valid fracture toughness evaluation. But, a valid concrete fracture toughness test normally requires very large size of concrete samples. Nevertheless, the recently developed ORNL Spiral Nottch Fracture Toughness Test (SNTT) approach has been successfully applied to concrete mortar samples with a 1.5 inch diameter rod samples, as illustrated below.

(see request folder for illustration)

SNTT mortar sample exhibiting classic tension brittle fracture characteristic

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

(1) In the analysis of metal failure it is usually sufficient to consider the stress in one dimension relative to the ultimate tensile stress of the material involved. The equivalent analysis of concrete failure is more complicated. It involves three dimensional stresses both in tension and compression. It is further complicated by dimensional changes such as shrinkage and creep. PII attempted to use conventional finite element analysis codes and were unable to simulate the observed delamination without implementing six methodology improvements. The PII model is a continuum, plasticity based, damage model for concrete. It assumes that the main two failure mechanisms are tensile and compressive. It includes:
1) the Hillerborg (1976) fracture model which combines fracture energy, tensile stress, and deflection angle to determine the onset of cracking in concrete. Research has found that tensile stress alone is not enough. Cracks will slowly propagate in conditions below the tensile capacity if sufficient fracture energy is available. For instances of acute over-stress the tensile capacity of the concrete is the most important factor. Because concrete is neither purely ductile nor purely brittle, the response to event specific conditions will vary. The dialation angle of a given type of concrete is a measure of the degree of ductility in the concrete and is what the model uses to simulate concrete response. Hillerborg defined the energy needed to open a crack of a unit area as the material property fracture energy, Gf. The model relies on stress-displacement rather than stress-strain. The yield function of Lubiner et al (1989) with modifications proposed by Lee and Fenves (1998) to account for different evolution of strength under tension and compression is used in the model.
2)modeling of visco-elastic effects of concrete creep, in conjunction with the fracture energy based fracture thresholds, to account for stress reversal effect (ie from compression to tensile)
3)use of a 360 degree, realistic whole containment modeling to reveal realistic containment displacements
4)incorporate the fracture creep phenomenon around the sleeves to account for a lower tensile strength near conduits (thus initiating small cracks which could be delamination initiation sites
5)use of fine meshes (around 1.0 cubic inch mesh) to reveal local stress concentration for crack initiation
6)the use of a variable elasticity and fracture toughness based upon local strain.

(2) Dialation angle of 23 degrees is used.
Flow potential eccentricity is a small positive number that defines the rate at which the hyperbolic flow potential approaches its asymptote. The default value of 0.1 is used.
The default value of 1.16 is used for the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress.
We assume a compression strength of 5500 psi but we do not see any failure due to compression.
We use 450 psi and 0.4 lbf in/in2 for the detailed sub-models.
We use the displacement formulation with direct cracking displacement off 0.002 in.

References:
Hillerborg et al, "Analysis of Crack Formation and Crack Growth in Concrete by Means of Fracture Mechanics And Finite Elements", Cemente and Concrete Research vol 6, pp 773-782, 1976

Lee, J and G L Fenves, "Plasti-Damage Model for Cyclic Loading of Concrete Structures," Journal of Engineering Mechanics, vol 124, no 8, pp 892-900, 1998

Lubliner, JJ et al, “A Plastic-Damage Model for Concrete”, International Journal of Solids and Structures, Vol 25, pp 299-329, 1989

(3) ABAQUS specifically analyzes for shear and found it was not a significant contributor to the event.

(4) Originally PII was brought in to analyze the root cause of the containment cracking. For that reason, PII developed a model which had damage sensitive elements only in the delamination area. Later on, PII expanded their model so that it could predict concrete damage in general (based upon damage energy) so that the model is no longer restricted to predicting delamination. The cohesion bonding assumed are tensile capacity and fracture energy.

(5) PII agrees with your comment about fracture energy. The code now uses the experimentally determined value of 0.40 lbf-in/in2. We overcame the issue of mesh size by developing an additional sub-model with a 1” cube mesh size. Perhaps more discussion is needed on the role of the vertical tendons. The comment above indicates de-tensioning vertical tendons induces significant tensile stresses on the outer wall. Another participant indicated he felt having tensioned vertical tendons create very significant tensile stress. When the containment was built the vertical tendon sleeves were generally straight. During tensioning, however, the wall moved inwards and the vertical tendon experienced a curved path and so exerted a straightening (tensile) effect. The vertical tendons are located inside the horizontal tendons in the containment wall so that the compressive stress supplied by the horizontal tendons generally prevents the overall effect of the vertical tendons from being positive strain. However, in an area near a vertical tendon and away from a horizontal tendon, the tensile effect of the vertical tendon can be significant. It is important to remember that the computer deals with this interaction as an integral part of the model solution so separating stresses from one tendon or another may facilitate understanding by the reader but it calculates delamination or not based upon the overall interplay of all the code elements. We just completed a parametric study of stresses in preparation for the re-tensioning effort. We looked at two different locations in the high stress areas of the steady state normally tensioned containment. One was next to a vertical tendon but far from the hoop tendons and the other was near hoop tendon but far away from a vertical tendon. The table below identifies the maximum stress components seen in the two locations just prior to the SGR outage.

(see request folder for table)

As you would expect, the vertical stress is most compressive near the vertical tendon and the radial stress is most positive near the hoops. The NRC question about the relative part played in the delamination by the vertical and horizontal tendons can be obtained from the comparison above. The hoops play the more significant role as would be expected given the observation of the delamination crack running from hoop to hoop rather than from vertical to vertical.
Notice that the radial stresses in normal operation would be considered low when compared to the tensile capacity of about 500 psi. This would encourage the view that radial stresses are too small to cause a problem The issue only becomes visible in evaluating the de-tensioning sequence.

PII agrees with the assessment that this is a very complicated analysis and that simplifications such as use of single unchanging material parameters may impact the results. PII found the model to be sufficiently accurate to establish the root cause of the delamination event but that additional accuracy is needed to ensure re-tensioning is successful. PII is pursuing time dependent spatially dependent parameters such as elasticity to reflect the complex interactions that do exist. This includes quantifying and addressing micro-cracking as it impacts parameters such as elasticity, tensile capacity, and compressive strength. A series of tests are being planned to determine the impact of cyclic stress on concrete strength parameters. The root cause analysis was clear based on the model available at that time. Additional testing is being performed in support of the retensioning effort because of the tight margins which exist in that upcoming evolution.

Misc Notes:

Response By:	Charles Williams		
Reviewed By:		**Date Response Provided:**	7/27/2010
Status:	Closed	**Date Closed:**	8/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 162

Individual Contacted: Charles Williams

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: As stated in PII report,

"Immediately after the delamination event, PII found that industry standard computer codes provided solutions which showed significant 'margin to delamination' when applied to CR3. This was caused by a variety of issues such as mesh size assumed, the fracture model used, and the treatment of creep effects. As an interim measure until more accurate modeling techniques could be developed, PII relied upon NASTRAN and its linear-elastic model to calculate local conditions and then ABAQUS to evaluate the local conditions and determine if damage resulted. However, due to limitations in the model, it was necessary to assume a modulus of elasticity of 1.1E6 psi throughout the structure. While this assumption allowed the model to predict the onset of delamination it had significant uncertainty. .. PII now uses an ABAQUS Global model using a visco-elastic fracture model and a detailed sub-model for decreased mesh sizing to provide accurate stress predictions. There are four input values which can be adjusted to make the computer model output match the benchmark data."

Those input parameters are listed below:

Parameter Typical value ABAQUS Global/Detailed Impact
Tensile capacity 500 psi 108/360 psiOnset of cracking
Fracture energy 0.40 lbf/in. 0.08 lbf/in.Onset of cracking
Elasticity modulus 3.45 E6 psi3.45 E6 psiRadial displacement
Creep coefficient 2.2 2.2 Radial displacement

It is interesting to note that typical or measured data are much larger than those used as input parameters for the FEM analyses, such as Fracture energy and tensile capacity. This in turn means that the damage criteria used in the PII simulation are less than that of the CR3 concrete failure strength/capacity.

In another portion of the report, PII provides some computer model plots having the following inputs that were based upon testing and modeling.

Figure Model E0 psiE1 psiF't psiG't lbf/inCreep
7.2, 7.3, 7.5, 7.6, 7.7, 7.8, 7.10 ABAQUS Global 3.45 E63.45 E6NANA2.2
7.12, 7.13, 7.15-7.24 ABAQUS Global Bay sub model 3.45 E63.45 E61080.082.2
7.4,7.9, 7.11 ABAQUS Global 3.45 E61.10 E6NANA2.2
7.15,7.26,7.27,7.28,7.29 ABAQUS Global Detail sub model 3.45 E63.45 E63600.082.2

"PII does not use NASTRAN to quantify concrete fracture so fracture parameters are not applicable. ABAQUS applies creep correction only for long time period intervals such as the time step of 30 years of operation with the containment tendons tensioned. E0 is the elasticity modulus and E1 is the creep adjusted elasticity modulus. F't is tensile capacity and G't is the fracture energy. 3.45 E6 psi is the average modulus measured on 22 CR3 containment cores. Fracture energy was measured to be 0.40 lbf/in. ABAQUS Global does not calculate fracture so those parameters are Not Applicable for it."

Based on the above statements, the input parameters for the PII Code (which should be the material property data used to properly characterize the behavior of CR3 containment concrete) were based on fitting parameters to make the computer model output match the CR3 benchmark data. This approach can have an affect regarding determination of the root cause of the CR3 delamination event since the benchmark data represents the values resulting from the complex interaction of detensioning, concrete removal in conjunction with the hydrodynamic impact force, and the pre-existing prestress history. Prior loading history may have contributed to cracking in the concrete containment before the detension events. Without an analytical simulation that utilizes the true material properties to describe the damage behavior of the aging concrete, the true evolution of paths of the containment concrete deformation that resulted in the final CR3 wall delamination may not be realized.

Therefore, if the size dependence cannot be reasonably resolved by the PII Code, then the delamination evaluation should not be pursued until the finer (or appropriate) mesh has been reasonably considered/developed in submodels. Thus, the reviewer has suggested alternative approach in dealing with this subject in the earlier section.

Background

Young's Modulus evaluation:

The relationship between compressive strength (fc) and modulus of elasticity (Ec) has many different expressions in the literature and codes. Using these empirical relationships, the CR3 concrete would be expected to have Ec in the range of 4.75 to 5.25x106 psi. However, the measured Ec was substantially lower, averaging 3.4x106 psi. This can be explained by a lower than normal modulus of elasticity for the aggregate.

Based on MPR report, it shows that the elastic modulus ranges from a low of Ec.avg.m = 3.29 × 106 psi to a high of Ec.orig2=4.67× 106 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: Ec.orig1 =4.0 × 106 psi. This elastic modulus is for the original concrete and was considered to be constant from that time to the end of plant life.

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

For the retensioning analysis, PII is using the existing data to develop time and spatial relationships for important parameters such as elasticity and tensile capacity. CR3 took a total of 22 core samples that were tested for elasticity. The result was 3.44 +/-0.34 Mpsi. It was on the basis of this large population of containment samples that PII chose the elasticity value that it did. However, we do realize that use of a single elasticity value is a simplification that was sufficiently accurate to model the delamination and demonstrate the root cause but for the re-tensioning sequence more accuracy is needed. For that reason we are continuing to refine our assumptions about elasticity and concrete strength over time and under stress.

Misc Notes:

Response By: Charles Williams

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Reviewed By:		**Date Response Provided:**	7/27/2010
Status:	Closed	**Date Closed:**	8/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 163

Individual Contacted: Charles Williams

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: It is not clear to the reviewer that the PII model makes an attempt to include analytical prediction of the reduction in the elastic stiffness caused by inelastic straining of the prestressed concrete. Nevertheless, it is likely that—even in such cases—the stress trajectories will not be entirely radial and the model must predict the response in such cases in a reasonable way. An isotropically hardening "compressive" yield surface can form the basis of the model for the inelastic response when the principal stresses are predominantly compressive. In tension, once cracking is defined to occur (by the "crack detection surface" of the model), the orientation of the cracks is stored and oriented, and damaged elasticity is used to model the existing cracks.

The consequence of tendon stress relief (due to inelastic straining of concrete or inelastic relaxation of steel tendons) is the reduction of the creep strain due to the decreased compressive stress in the concrete wall.

In a tendon detension event, the compressive stress relief from the tendon conduit can also assist the tensile and shear stress fields' development in the concrete around the tendon conduits. The combined effect of increased tensile stress due to tendon detensioning and conduit relaxation can be a major driver for the crack initiations and growth around/along the conduit region.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The reviewer is correct. Over time the concrete creeps but the conduits, liner, and rebars are mostly deforming elastically. During de-tensioning the conduits are putting the concrete in tension due to creep differential between concrete and steel. We model this condition as the reviewer suggests. Stress relief is modeled to include reduction in tendon load based upon measured values and based upon creep. The original model for detensioning and root cause investigation had a discrete step in the modeling process after initial tensioning to implement creep over the 30 year operating time. Additional creep was not considered for the subsequent steps of detensioning. Including creep in a calculation impacts the effective modulus of elasticity used in a specific activity. For long time periods such as 30 years of operation it is appropriate but for short duration activities creep does not play a significant part. With the additional of visco-elastic modeling the application of creep became automatic. The addition of the damage plasticity model required going back to manual application of creep.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 7/27/2010

Status: Closed

Date Closed: 8/10/2010

20-Apr-11 7:56:38 AM

Request Number: 164

Individual Contacted: Charles Williams

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: Does the PII code consider the development of a concrete cracking algorithm that is associated with a reduction in shear strength and stiffness? It is prudent to provide a procedure in the PII code to model these reductions as cracks develop. As cracks widen the shear contribution from aggregate interlock under the action of external loads (e.g. prestress or detension events) and then reinforcement (if present) through dowel action of rebars (or in a similar way of the tendon conduits), can become of interest.

It is important to point out that the crack phenomena are very dependent on the opening of a crack as well as the transmitted compressive stress that accompanies the shear displacement. The relation between compressive stress and shear displacement is normally defined as dilatancy. Generally, the shear strength attributed to aggregate interlock can be modeled by using discrete crack formulations, based on empirical research findings that capture crack dilatancy. It is also expected that the weaker shear strength due to weaker aggregate interlock resulting from a weaker aggregate can also affect the cracking evolution history in the FEM modeling. In general, three stages in the cracking process can be distinguished:

(1) The linear-elastic state;
(2) The development state in which a tension-softening model is used; and
(3) The open-crack state, for which the crack-dilatancy models can been implemented into FEM analyses.

Warping can occur in the anisotropic shear loading redistribution condition, such as in a cut-open ring, piping, channel bean, where the resultant shear force does not act at the shear center of a structural component. For instance, in CR3 detensioning event, detensioning of the 10 vertical tendons can produce significant distortion stress redistribution in the containment structure due to non-symmetrical loading, as well as significant distortion from 17 hoop tendons that were detensioned sequentially from the bottom up to the top of the SG opening. Therefore, a true sequential detension simulation is vital for a clear judgment related to the crack initiation/growth potential of the CR3 detension-induced delamination event.

Moreover, the sudden release of a tendon generates an oscillating tensile wave through the horizontal tendon conduit, as well as transmission/propagation of shear waves from the vicinity of the tendon anchor sites to the concrete containment.

Background: On subject related to shear flow

Unrestrained warping accompanies twisting of the section, when shear forces are applied away from the shear center. Similarly, the shear flow can also be induced by unbalanced axial compressive stress due to vertical tendon detensioning, as well as unbalanced circumferential compressive force from the hoop tendon detension events. The shear flow along the axial direction of the wall induced by cutting the axial (vertical) tendons or concrete is illustrated in the figure below to the right.

(see request folder for illustration)

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

Yes, shear strength and stiffness is reduced based on the development of a crack. The evolution of the strength and stiffness are implemented in Abaqus Concrete Damaged Plasticity model for the general multiaxial stress conditions based on Lee and Fenves (1998). The model assumes that the elastic stiffness degradation is isotropic. If damage occurs the reduction in stiffness is applied in all directions. As explained by Juan Hurtado (the person who implemented the Concrete Damaged Plasticity model in Abaqus) this implementation is an approximation but not a serious issue as long as the mesh is fine enough to resolve the narrow crack bands. In the CDP model a crack is represented as a narrow area of strain localization in the mesh. Such areas of strain localization cause the overall structural stiffness to become anisotropic.

Misc Notes:

Response By: Charles Williams

Reviewed By: **Date Response Provided:** 7/27/2010

Status: Closed **Date Closed:** 8/10/2010

Request Number: 165

Individual Contacted: Charles Williams

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: As provided in PII report,

"On 9/26/09 the Crystal River steam generator replacement (SGR) outage began. It included the creation of an access opening in the containment wall. The construction opening was a 27' by 25' rectangle running from elevation 183' to 210' centered on the centerline of bay 34. From 9/27/09 to 10/1/09 plasma cutting of 17 horizontal and 8 vertical tensioned tendons occurred. Two additional vertical tendons were de-tensioned with jacks for possible reuse."

The PII computer simulation is not a tendon-by-tendon scenario. Instead, it takes the total effect of de-tensioning all the selected tendons and divides it into twenty steps so each frame corresponds to a 5% change in all of the tendons rather than 100% of one tendon. This is not consistent with that of the CR3 detensioning sequence. Moreover, the vertical tendons are located off the center and close to the outer wall surface. This will generate higher compressive stress on the outer concrete wall region due to bending induced flexural compressive stress, in addition to the compressive stress by tensioning vertical tendons. Regarding the hoop tendon detensioning sequence, it was carried out from the bottom of the opening panel and then progressively moved up to the top of the SG opening panel. Such progressive upward detensioning can certainly affect and create a unique crack initiation/propagation contour in the concrete region. Thus, the stress redistribution and its impact on crack initiation depend on the individual tendons and their detensioning sequence.

Based on the current PII detensioning simulation approach, it may well overestimate the detensioning induced delamination event in the concrete containment. This is because the stress relief resulting from an individual tendon detensioning event can be effectively transmitted to the nearby neighboring region without generating a significant disturbance (or large area global deformation) to the concrete media, thus reducing the cracking potential in the concrete media. It would be very beneficial to further clarify this if PII would carry out a more realistic simulation that can mimic the exact sequence of CR3 detensioning events.

Furthermore, the plasma cutting of the tensioned tendons can certainly introduce dynamic/impact forces into the system that may further assist the crack growth within the concrete containment. It is not clear to the reviewer what type of kinetic model was used in the PII dynamic/impact analyses related to the tendon plasma cutting induced shock simulation; for instance, what was the damping ratio and the rate of loading used in the vibration impact analyses. Rate dependent tensile strength of concrete is another issue that may need to be considered/addressed.

The tensioned vertical tendons actually form a first line of defense to compress the containment wall, with higher compressive stress to the outer portion of the concrete wall; and the tensioned horizontal tendons form the defense to provide the radial constraint. Therefore, upon the initiation of vertical tendon detensioning alone, due to the Poisson effect, and the tensile force field induced by the vertical conduit, elastic recovery can effectively produce cracks around the conduit in the concrete, while the hoop tendon remains intact.

Therefore, the overall effectiveness/accuracy/reliability of the PII simulation technique of simultaneously detensioning all the 27 tendons in a progressive manner in determining the root cause of CR3delamination event is questionable and needs more detailed examination.

Background

The PII CR3 delamination analysis was based on a model with a gradual (ramp-up) detensioning of the entire 10 Vertical and 17 Horizontal tendons simultaneously. This is not consistent with the order used for the CR3 detensioning events: the majority of the vertical tendons were detensioned before the hoop tendons were detensioned, and the sequence was that the two centrically located vertical tendons were released first and then alternative detensioning proceeded around the central vertical tendons; while the horizontal tendons were detensioned at the bottom of SG opening first and then progressed to the top of the SG opening. Furthermore, the plasma cutting was used in the cutting the tendon wires which will introduce/develop dynamic impact loading to the concrete structure.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

We did not simulate tendon-by-tendon SGR de-tensioning. However, we acknowledge that the propagation of the crack may have been dependent on the tendon-by-tendon release to some extent. One finite element analysis was performed that considered the tendon-by-tendon SGR de-tensioning. The result suggested that the highest stress state occurred at the completion of the SGR de-tensioning. Also, the symmetrical hour glass shape of the SGR delamination crack suggests to some extent that tendon-by-tendon release sequence was not important in the final damage.

The sudden transient release of tendons are not considered in the cracking models. One failure mode considered the shock wave due to the sudden release of an entire tendon. It concluded that the stress due to the shock wave propagation and vibration of the structure were insignificant.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 7/27/2010

Status: Closed

Date Closed: 8/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number:	166		
Individual Contacted:	Charles Williams	**Date Contacted:**	7/13/2010
Requestor/Inspector:	Louis Lake	**Category:**	Issue

Request:

U. Colorado testing showed that the creep coefficient for the cored CR3 concrete samples is 2.39. The PII approach uses the calibration process to “guess” the creep coefficient as 2.2 and by using a creep coefficient of 2.2, the effective Young's modulus for the PII creep evaluation is 1.1*106 psi. The estimated creep coefficient based on ASTM C-512 standard is around 0.87.

The U. Colorado report uses the "time temperature superposition principle" (TTSP) approach that utilizes an accelerating testing protocol to evaluate the creep coefficient of CR3 concrete core samples. There are several issues, relating to the CR3 core samples and the TTSP approach used in U. Colorado's evaluation, which may have serious impact/consequence on the validity and accuracy of the estimated creep coefficient for CR3 concrete.

(1) Firstly, the superposition theory of the TTSP approach is only valid if the similar mechanism /trend (self similar/self consistent) is presented at all the temperature levels; this can be checked by determining the activation energy of the process which should be independent of temperature. However, the creep of concrete is known to be sensitive to various environmental and testing conditions, such as humidity and temperature, as well as stress. In the Colorado creep study, creep was accelerated by changing the temperature while other parameters are kept constant. The creep test results are shown in Figure 1. Clearly, different trends, that are not self similar, were observed for creep testing done at 40 and 80°C. This violates the validity requirement of using TTSP approach.

(see request folder for Figure 1)

(2) Secondly, based on the size and specimen number requirement specified in ASTM: C 512 – 02 Standard Test Method for Creep of Concrete in Compression, which specifies that
Specimen Size—The diameter of each specimen shall be 6-1⁄16 in., and the length shall be at least 11-1⁄2 in.
Number of Specimens—No fewer than six specimens shall be made from a given batch of concrete for each test condition; two shall be tested for compressive strength, two shall be loaded and observed for total deformation, and two shall remain unloaded for use as controls to indicate deformations due to causes other than load. Each strength and control specimen shall undergo the same curing and storage treatment as the loaded specimen. Accompanied unloaded samples; the length change of these samples are measured and subtracted from the loaded samples to determine creep by load. This correction is intended to eliminate the shrinkage and other volume change.

However, there were only two sub-size creep test samples, with ~3-in diameter, that were used by the U. Colorado to develop the creep coefficient. Due to the smaller radius (thus, higher slenderness ratio) compared to that of the ASTM specification, the potential of buckling related instability or bending during the creep testing is significantly higher than that of using the ASTM Standard creep test sample. Furthermore, based on CEB-FIP Model Code (1990), the creep sample had a relatively small effective thickness (effective thickness is defined as area of cross-section divided by the semi-perimeter in contact with the atmosphere) resulting in a higher creep coefficient. In the U. Colorado case, the effective thickness of the sample was about half that of the ASTM Standard test sample size.

Furthermore, use of a small diameter sample can affect the mechanical test results, for instance, vibration induced surface crack, internal crack, or other damage to the core samples, which may further complicate the creep test result. Moreover, the elastic strain recovery for the core boring sample due to removal of the biaxial compressive prestress need to be benchmarked and corrected from the creep test, as described earlier.

The biaxial loading of prestressed concrete will generate less creep strain relative to that from a uniaxial test, as shown in Fig. 2, where the biaxial creep value is about 70% of that obtained from an uniaxial test.

(see request folder for Figure 2)

Therefore, a proper correction margin is needed to adjust the over-prediction of the creep values, and to cover other man-made sample damage potential due to collecting in-situ CR3 core samples, and potential eccentric loading during creep loading or heating-induced concrete structure/component property changes.

Background

The U. Colorado obtained creep strain of the concrete used in Crystal River NPP by two accelerated creep tests at 80 °C and 40 °C. Preparations of the specimens and the test system are described below

- Two cylindrical specimens (#99 and #102) were received for the creep tests.
- The two specimens were cut into 11.5 in. lengths.
- Both specimens were capped on top and bottom surfaces.
- Two contact points were installed on the top and bottom portion of each specimens. The distance between the two contact points equals the gage length of the dial gauge to be used to measure the length change of the specimens.
- The level of accuracy of the dial gauge is 0.0001 in.

The Experimental System

- An environmental chamber is mounted on the MTS loading machine.
- The MTS provides a stable compressive force at 27 kips (less than 40% of the compressive strength).
- The chamber maintains a constant temperature of 80 °C or 40 °C and a stable relative humidity (RH) in the range of 60%-70%.
- The force, temperature, and RH are automatically controlled by the experimental system.
- In the first day, readings were taken every 8 hours; and in the second day every 12 hours; and then every 24 hours to the end of test.
- Specimen #99 was tested under 80 °C for 11 days and Specimen #102 was under 40 °C for 14 days.

Creep Test Results of Specimen #99 under 80°C
Applied force 27 kips
Diameter of cylinder 3.7 in
Applied stress 2512 psi
Elastic strain (the first data point) 0.001045 in/in

Creep Test Results of Specimen #102 under 40°C
Applied force 27 kips
Diameter of cylinder 3.72 in
Applied stress 2485 psi
Elastic strain (the first data point) 0.0004667 in/in

The creep coefficient estimate based on ASTM C 512 standard is around 0.887, as described below.

(see request folder for chart)

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

There are two factors to consider. One is the magnitude of the creep effect and the second is the time dependence of the effect. Creep the slow increase of concrete strain over time when the concrete is placed under stress. Although the rate of creep slows down with time it continues even after 40 years of stress.

(see request folder for chart)

The acceleration due to temperature is assumed to be 200 times at 80 C so times listed for that sample are adjusted accordingly. The factor for 40 C is 14. These figures are several 4 times this simple activation assumption would give. It is in accordance, however with the data available on page 206 of "Creep and Shrinkage in Concrete Structures".

The agreement between the accelerated samples and with ASTM C 512 is reasonable considering the process being used to obtain data quickly. It would be a mistake to assume the value of creep at 28 days is the final value. Structures continue to see creep after 40 years of tensioning The creep coefficient used by PII is 2.2. That translates into a total strain to elastic strain ratio of 3.2. Remember that creep is used for only one stage in the analysis process. That is the step which inserts 33 years of creep growth between 1976 and 2009. The Temlin Nuclear Plant has been monitoring their containment creep for the past 40 years and the total strain to elastic strain they now have is 2.83 or a creep ratio of 1.83. Clearly using the ASTM value of 0.88 at 28 days is not generally applicable to a building that is decades old. Extrapolating the 1970 room temperature creep data to 35 years
gives a total strain ratio of 32 and a creep ratio of 2.2 In terms of the root cause analysis the time dependence is relatively insignificant and only the magnitude is important..

Another approach is to plot the total strain ratio for a variety of conditions including set time, time under tension, and temperature. This is shown in the graph below:

(see request folder for graph)

Notice the temperature related data fits nicely into the overall scheme.

PII based its selection of creep coefficient on three considerations. The plant specific testing was one factor. A second factor was the tension loss observed in tendon surveillances which were translated into an effective creep coefficient causing the detensioning. A third factor was a literature search for expected creep coefficients seen elsewhere. PII did a parametric study on the sensitivity of delamination to the creep ratio assumed. They found significant damage with a creep ratio of 1.5. The conclusion of the root cause of the delamination event is not impacted by varying the assumption of creep. However, the re-tensioning of containment will require as accurate a model as we can develop so PII is actively reviewing the available creep data. The most sensitive test of creep comes with benchmarking radial displacements and tendon tension degradation. These sources of comparison will be used to develop a creep/creep relaxation model for re-tensioning.

The stress applied in the UC testing was about 2500 psi. This is far higher than the stress seen while the samples were in containment (100 psi). The results of the 1970 creep tests at SR3 also support the results of the

UC testing.

The ultimate proof has to be benchmarking of the radial displacement observed over the 35 year creep period. PII is in the process of re-evaluating absolute radial position as measured by the laser measurements to support the retensioning analysis..

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 7/27/2010

Status: Closed

Date Closed: 8/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 167

Individual Contacted: Charles Williams

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: As stated in PII report,

"Analysis of concrete strength records lead to the conclusion that aggregate strength limited concrete strength development. Grout cores (Concrete without the coarse aggregate that used to start each pour) tested significantly stronger than concrete from the same pour. These observations indicate that the presence of aggregate limits the tested strength of the concrete. This conclusion is further validated based on the observation of core fracture – where all cracks propagate through the aggregate and not around the interface zone."

Therefore, the soft aggregate has a significant impact on the overall concrete properties, including the cracking behavior. Crack bridging occurs when cohesive stresses join the opposite faces of a crack, shielding the crack tip from the full effect of the applied load and thus giving rise to increased fracture resistance. Cohesive stresses are present not only in cementitious materials, but also in ceramics and are caused by reinforcing fibers, particles (such as aggregates or inclusions), or simply regions of microscopically irregular crack surfaces causing topological interference. Cohesive stress models were first suggested by Dugdale (1960) using a constant stress field near the crack tips. Furthermore, Dugdale's model uses a through thickness crack configuration and is in a plane stress condition; where the associated thickness constraint effect has not been extensively investigated in concrete fracture mechanics.

The cracking bridging/cohesive stress is the basis of the Dugdale model fracture criteria (as well as in Hillerborg's damage model) and is widely used in concrete fracture mechanics. For a weak aggregate, the crack bridging effect is significantly reduced, due to stronger matrix in relation to the weak inclusion. Furthermore, the embedded crack configuration of the CR3 containment that does not resemble the through thickness (plane stress) crack configuration of Dugdale (cohesive model) approach. Therefore, the effectiveness or correctness of the cohesion model used in describing concrete fracture behavior for CR3 delamination investigation can be questionable. Moreover, in the absence (or significant reduction) of these cohesive stresses due to weak aggregate, we would have just simplified a brittle material, instead of a more complex quasi-brittle material. Thus, a more simplified approach may be warranted to describe CR3 concrete damage behavior.

Multi-axial effects: multi-axial confinement leads to an apparent increase in the compressive strength of concrete. Therefore, multi-axial compression in some degree can also mitigate the deficiency/degradation of weaker aggregate used in the CR3 containment construction.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

To the extent it is possible to do so, PII uses experimentally verified parameters. This includes F'c, E, and F't. To further refine the PII model in support of retensioning, additional testing is being arranged to investigate the impact of deep cycle fatigue on concrete. This is a more reliable method to determine model parameters where it is possible to do so.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 7/27/2010

Status: Closed

Date Closed: 8/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 168

Individual Contacted: Charles Williams

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: As stated in PII report, "The pre-existence of small cracks emanating from the tendon sleeves in the plane of the hoop tendon sleeves has been used in the PII modeling and it has been used in the PII uncertainty analysis in the Margin-to-delamination calculation. It is likely that such non-active cracks would be present in other Plants not analyzed here because the stresses calculated using linear elastic mechanics (NASTRAN or ANSYS basic calculations) are very large and would exceed the tensile strength of all concrete materials. It is important to realize that these are structural cracks (not micro-cracks) and they will not allow transfer of stresses across the crack opening. It is also important to realize that theses cracks are non-active, meaning that the localized high stresses are eliminated by the cracking. With that said, tendon sleeve hole locations and hoop planes generically possess higher potential for crack propagation if exposed to additional sufficient drivers, due to this "weakening" of an already reduced effective concrete thickness between the ducts."

I concur with the PII observation that there exists a high potential of cracks around the tendon sleeve regions. However, these cracking regions more likely happened during the tendon tensioning event or due to material mismatch between concrete and steel conduit due to other environmental effects, such as temperature or/and thermal cycling (due to power outage or reactor shut-down and start up thermal cycling), that occurred before the CR3 detensioning events. The existence of these high crack density regions should be taken into account and integrated into FEM modeling/analyses, due to its weakening phase and the high probability for developing/forming cracks and further assisting the crack propagation along the tendon sleeves.

However, it is not clear to the reviewer that these regions were properly taken into account in PII delamination simulation analyses.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

In order to support the retensioning program, PII is correlating plant specific core data to address the issue of degradation of material properties over time. Thus the issue above will be captured in the variable parameter approach being developed.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 7/27/2010

Status: Closed

Date Closed: 8/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 169

Individual Contacted: Charles Williams

Date Contacted: 7/13/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request:

As stated in the PII report,

"Following is a summary of the failure scenario. Seventeen tendons from elevation 183' to 210' are de-tensioned. Ten vertical tendons, centered on azimuthal angle 150 degrees, are also de-tensioned. Generally the vertical tendons were de-tensioned first. This creates an imbalance in the pre-stress loads on the containment building and a bulge develops in bay 34 centered on elevation 196' and azimuthal angle 150 degrees. The bays on the opposite side of the building curve inward. The analysis also indicates that initial tensioning causes more inward displacement of the middle of a bay than around the edges. No displacement scans were performed during the SGR de-tensioning but the plant remained in the partially de-tensioned condition until March, 2010 when it was largely de-tensioned for delamination repair work. The scope of the repair de-tensioning was to de-tension essentially all horizontal tendons in all bays from H17 to H44 (elevation 152' to elevation 239'). This de-tensioning was accomplished in eleven steps from pass 1 to pass 11. Laser scanning was performed after each pass from pass 4 to pass 11. Thus, the laser scan only captured part of the de-tensioning but it does afford a comparison between the expectations of the scenario and the actual response of the plant. The laser data is presented as the radial displacement change using the un-tensioned condition after pass 11 as the baseline, so outward motion is negative and inward motion is positive. The point-by-point detailed comparison is discussed in Attachment 4."

From the laser scanning data, it indicates that the PII predicted relative radial displacements are in general less than that of the laser scan data at different locations, especially at the SG opening regions. This interesting phenomenon may indicate that the PII model may underestimate the elastic recovery strain due to detensioning, or result in an inaccurate estimate of the creep deformation (strain) in the PII FEM model development. It is noted here that a high Young's modulus (~3.4 E6 psi) was used by PII in this portion of the creep deformation analysis, as compared to Young's modulus of 1.1 E6 psi that was used in the earlier creep deformation analyses.

Background

The overall strain can be described below.

(see request folder for description)

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

PII is currently reviewing the benchmark data with the intent of establishing optimal time dependent material properties which will address the differences between specific benchmarks and the analysis code.

There are several benchmarks to consider in selecting an elastic modulus. During the 1976 Structural Integrity Test creep was not an issue due to the short duration of the test. Using 3.45 Mpsi for E, the radial displacement predicted by Abaqus matched the observed radial displacements up to about elevation 170' and then tended to slightly over-predict the displacement above that elevation. During the laser scanning in 2010, Abaqus under-predicted the radial displacement. PII is developing a time and spatial dependence of material properties to address these findings. However, the modulus used by PII in the root cause evaluation is fully supported by the 22 core sample material property tests.

Misc Notes:

Response By:	Charles Williams		
Reviewed By:		**Date Response Provided:**	7/27/2010
Status:	Closed	**Date Closed:**	8/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 170

Individual Contacted: Charles Williams **Date Contacted:** 7/13/2010

Requestor/Inspector: Louis Lake **Category:** Issue

Request: Normally, the hydrolasing process utilizes a variety of remote-operated rotating water jet tools. These tools direct ultra high pressure water (20,000 psi or above) at velocities up to twice the speed of sound to remove material coverings from overhead, vertical, and horizontal structural surfaces.

The cavitation damage induced by hydrolasing can effectively pit the target surface and remove the material from the impact area. Based on our cavitation research, the cavitation pressure at around the 100 psi range is sufficient to generate the surface pitting damage of stainless steel samples at a relatively short time frame, as illustrated below. Therefore, the higher pressure water jet can certainly produce even more significant pitting damage to the concrete material, especially around the existing crevices of the concrete material. The cavitation induced shock wave and the trapped localized water jet current can effectively assist the crack initiation, and crack growth/propagation.

Therefore, the PII conclusion that Hydro-demolition did not cause cracking and degradation of the concrete adjacent to the demolition area may need a closer look and a more thorough examination.

(see request folder for illustration)

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

Observations during hydrolasing confirmed that the process will enlarge existing cracks. Thus the technique can increase the observed crack size, but the technique is generally considered to be the least traumatic to the concrete surface when it is intact. PII still feels that the cracking occurred due to the sequence and scope of tendon de-tensioning which is the crux of the delamination issue. The root cause report did investigate the potential for delamination due to the hydrolasing process and concluded that it was not the cause of the delamination. Cracking was observed almost as soon as the delamination region was uncovered on a test basis. There was a very different concrete residue in the delaminated region. The concrete came off in large chunks rather than small aggregate that occurred above and below the delaminated region. That indicates the delaminated area was a pre—existing condition rather than being caused by hydrolasing.

(see request folder for photo)

Notice the very smooth nature of the eroded surface exposed by hydrolasing.

Compare the surface exposed in the area of delamination (about 10”deep)

(see request folder for photos)

At this point the hydrolasing is below the delamination zone and the smooth erosion surface has returned. The nature of hydrolasing does respond to the existence of cracks but these photos indicate the basic cracked condition is not caused by hydrolasing.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 7/27/2010

Status: Closed

Date Closed: 8/10/2010

20-Apr-11 7:56:38 AM

Request Number: 171

Individual Contacted: Paul Fagan **Date Contacted:** 7/21/2010

Requestor/Inspector: Louis Lake **Category:** Issue

Request: CAP Report:

1. In numerous places, both the CAP report and the Attachment 1 (PII report) make reference to the SGR construction opening, specific vertical and horizontal tendons (e.g. 34V12, 53H27, etc.) within the opening, and describe the scope/sequence of detensioning which is also identified as the primary root cause for the delamination. However, there is no figure or drawing included in the main body of either of the reports that show the location and dimensions of the SGR opening and the locations and designations of the tendons within and around the opening and other such pertinent information (if any). Such a figure or drawing is fundamental information for this root cause report to stand alone and allow ease of understanding of the problem and the root cause investigation to a third party reader. Since the CAP report recommends sharing the report as both internal and external operating experience, suggest that such a figure or drawing be added and identified in the main body of the report.

2. Last but one paragraph on page 10 under "Previous Operating Experience (Internal and External)" discusses the operating experience with regard to the dome delamination at Crystal River Unit 3. Since the fragility of the coarse aggregate is a very important characteristic inherent in the CR3 containment concrete that must be recognized as a fundamental weakness in the CR3 containment operating experience, suggest adding the following important fact from the conclusions of the dome delamination report at the end of the paragraph: "The complete fracture of the coarse aggregate on the delaminated surface and the variations in tensile strength values obtained from the direct tensile tests indicate that the fragility of the coarse aggregate permitted local cracking to propagate." Also, suggest change the second and third sentences of the second paragraph on page 10 to include "fragility of the coarse aggregate and FM 3.4" to read: "Several of the factors, such as high radial tension, stress concentrations, and lower than normal concrete tensile strength, and fragility of the coarse aggregate, were identified as consistent contributors. These causal factors are discussed in more detail in FM 1.2, FM 1.5, FM 2.12 and FM 3.4, respectively."

3. Since it is as important to perform a detailed analysis of retensioning as for detensioning, the phrase "…prevent detensioning more than one tendon/group without…" in Item 3 under the title "Summary of Corrective Action(s) to prevent Recurrence (CAPR):" on page 3 of CAP Report should read: "…prevent detensioning and subsequent retensioning of more than one tendon per tendon group without…". This comment also applies to 1st row, 6th column, of the second item of the corrective action plan on page 15.

4. On page 4 of CAP Report, first sentence in last paragraph, since the dome is also a shell (spherical), suggest changing the phrase "The delamination of the containment shell is unprecedented, …" to "The delamination of the containment cylindrical shell is unprecedented,…"

5. In the first sentence of the last paragraph on page 11, add the words "and the dome" after the words "…all six bays" since the dome is also susceptible to delamination.

6. On page 14, the first sentence in the first bullet should read: "The high hoop and vertical prestress also translates into high…"

7. On page 14, fourth bullet, the description should read: "The coarse aggregate used in the containment concrete was relatively soft, porous, and susceptible to tensile fracture and

propagating cracks."

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

1. Figure 1 added that shows the SGR opening location and size as well as the tendon numbering scheme at the SGR opening.

2. Added fragility of course aggregate and FM 3.4 to list of common contributors. This discussion provides the relevant comparison of contributors.

3. No change. The use of detensioning in this text is more the condition of being detensioned, not the act of detensioning. It is recognized and understood that when there is a need for detensioning, that retensioning is also part of the process, so the same type analysis is required. The CAPR for the current repair was separated into analysis for detensioning and analysis for retensioning since the analysis for detensioning for the repair has been completed vs the analysis for retensioning that has not yet been completed.

4. The wording changed from" shell" to "wall".

5. Wording changed as recommended.

6. Wording changed as recommended. The text has been more focused on the effect of the horizontal hoops. The vertical tendon prestress also contribute to the stresses in the concrete.

7. No change. The text as written is correct. The FM provides additional discussion of this topic.

Misc Notes:

Response By: Charles Williams

Reviewed By: **Date Response Provided:** 7/30/2010

Status: Closed **Date Closed:** 8/9/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 172

Individual Contacted: Paul Fagan

Date Contacted: 7/21/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: CAP Report:

1. The contributing failure modes to the root cause are listed on Page 8 (last but one paragraph), pages 13-14 of the CAP Report and elsewhere in both the CAP and PII reports. The title descriptions of the 8 supported failure modes seems not consistent in many places with that in the individual failure mode evidence sheets included in Attachment 6 to the PII report. Suggest that these failure modes title description all across the CAP and PII reports be consistent with those on the individual failure mode evidence sheets included in Attachment 6 of the PII report.

2. On page 16, last row, 6th column, the phrase "…during and retensioning to confirm…" should read "…during and after retensioning to confirm…"

References:

Response Assigned to: Charles Williams

Date Due to Inspector: 7/30/2010

Response:

1. Titles changed to be more consistent and better align with the titles on the FM titles.

2. Deleted the "and". Scanning will be done after each retensioning pass similar to how it was done for detensioning. It is not intended for scanning to be done continuously during retensioning.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/3/2010

Status: Closed

Date Closed: 8/9/2010

Request Number: 173

Individual Contacted: Paul Fagan

Date Contacted: 7/21/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: CAP Report:

With reference to the third row from the bottom of the Table on page 11, please confirm if ANO-1 and Palisades performed concrete removal by detensioning only the tendons within the SGR construction opening.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Design documents provided by ANO-1 showed that 4" of concrete removal was allowed after detensioning tendons within the SGR opening (Phase 1 and 2) consisting of 16 Horizontal Tendons and 6 Vertical Tendons. This was allowed once in Mode 5. Then concrete from 4" to 8" was allowed once the tendons were removed. Followed by concrete removal >8" depth after degreasing. Tendons outside of the opening were detensioned after defueled or >23' over fuel. Total scope of detensioning was 36 Vertical and 34 Horizontal Tendons.

Design documents provided by Palisades allowed concrete removal in three areas at bottom of the SGR opening to begin after 10 Vertical Tendons and 9 Horizontal Tendons in the SGR opening were detensioned. The concrete in the remaining area of the SGR opening was allowed to be removed once 15 Vertical Tendons and 35 Horizontal Tendons were detensioned. There are 13 Vertical and 31 Horizontal Tendons shown within the SGR opening. Total scope of detensioning was 69 Vertical and 60 Horizontal Tendons.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/3/2010

Status: Closed

Date Closed: 8/9/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 174

Individual Contacted: Paul Fagan

Date Contacted: 7/21/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: PII Report:

General Comment on PII Report: Figure 7.1 on page 29 is a repeat of Figure 2.1 on page 14. Also, Figure 7.15 on page 46 is a repeat of Figure 3.5 on page 46. Figure A4.8 on page 102 is a repeat of Figure 7.14 on page 45.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The comment is that there are a few repeat figures in the report. That was intentional to avoid having the reader hunt for figures for comparison. No changes are required.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/3/2010

Status: Closed

Date Closed: 8/9/2010

Request Number: 175

Individual Contacted: Paul Fagan

Date Contacted: 7/21/2010

Requestor/Inspector: Louis Lake

Category: Issue

Request: PII Report:

1. Page 19 of PII Report – The title for Figure 3.3 should be clarified to state: "Tensile and compressive stresses in concrete wall cross-section due to prestressing force in hoop tendons"

2. Page 19 of PII Report – The second paragraph cites normal average radial tensile stress values of 23 psi and 31 psi at the vertical plane of the horizontal tendons and compares it to tensile stress peak values of 1630 psi and 510 psi at the interface with the hoop tendon sleeve. The comparison made is between apple and oranges: The 23 psi and 31 psi are average radial tension stress values due to prestressing force in the hoop tendons only. However, the 1630 psi and 510 psi are the peak tensile stress values at the interface of the hoop tendon sleeves due to prestressing forces in both the hoop tendons as well as the vertical tendons (including its eccentricity). This should be explicitly clarified in the discussion. Also, it must be clarified that the stress contours shown in Figures 3.4 thru 3.6 on pages 21 thru 23 of the PII report are based on linear elastic analysis of an uncracked concrete section, and the actual stress contours will be different since the concrete would crack and redistribute the stresses when the stress exceeds the tensile capacity in such a way that the resulting stress distribution in within the tensile capacity of the concrete. If possible, suggest including corresponding figures showing stress contours after cracking when the tensile capacity of concrete is exceeded around the tendon sleeves.

3. Pages 20, 31 & 71 of the PII report discuss and tabulate measured material properties that were input into the fracture-based computer model for simulation of the delamination event. Please provide references (e.e. FM X.X Exhibit y) to the source of the measured input material properties. For instance, the measured values of F't, G't, creep on these pages were not found documented in any of the FM evidence sheets or exhibits.

4. Figure 7.10 on Page 41 of the PII report provides a detailed cut-away profile plot of radial displacement following the SGR detensioning. If possible, suggest including the same cut-away profile with corresponding principal stress contours that would directly show the corresponding state of stress in the containment wall following SGR detensioning.

5. Page 22 of PII Report, first paragraph, second sentence, please correct the sentence as: "The circumferential surface through the vertical centerline of the hoop tendons has the lease concrete surface area."

6. On page 42 of PII report, the fourth sentence should read "…impact of detensioning 17 hoop tendons and 10 vertical tendons in creating a double…" Likewise for the first sentence on page 43.

7. On page 56 of PII report, the table of detensioning chronology should state what type of tendons (vertical or horizontal) were detensioned on 9/27/09 thru 9/30/09, as was done in the first and last rows of the table.

References:

20-Apr-11 7:56:38 AM

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

Revised Response:

Comment 1 - Clarification added to referenced text regarding stress due to hoop tendon prestress vs stress due to combination of hoop and vertical tendon prestress.

Comment 2 - Additional clarification has been added. As indicated in the discussion, it is not intended to be a detailed discussion. The new wording is shown below:

"A horizontal tendon provides the radial compressive stress of about 340 psi averaged over the diameter of the tendon sleeve. The average over the entire cross-section of concrete being radially compressed by that horizontal tendon is about 100 psi. Normally the average radial tensile stress would be 23 psi but due to concrete displacement it is 31 psi at the centerline of the horizontal tendons.

The tensile stress peaks at a very high value right at the edge of the horizontal tendon hole at the intersection with a vertical tendon due to the vertical and horizontal tendons. It reaches 1630 psi at the hole and drops off rapidly by a factor of 2 about every inch away. More generally, tensile stress peaks at about 510 psi on the edge of a horizontal tendon away from the vertical tendons and drops off about a factor of 2 every inch away. Figures 3.3 to 3.6 show a general display of stress variations along various directions in the containment concrete assuming no cracking in the concrete. For exact values refer to the specific calculations of design parameters found in the applicable FMs and the figures in Section 7 of this report."

Comment 3 requests a reference for the measured material properties. FM 2.12 Exhibit 19 is a good summary of most of the measured properties. Creep is discussed in considerable detail in FM 4.5 with measured value identified in Exhibit 15. Gt is equivalent to Gf and was measured for this project at the University of Colorado. Attached is their report which includes Gf for sample 92A on page 4. Since this report was not included in the FM's it is being included as an Appendix to the PII Root Cause document.

(see request folder for attachment)

Comment 4 requests a figure with max principle stress as compared to Fig 7.10 which shows displacement. Figures 7.17 through 7.23 show max principle stress at the delamination layer as SGR detensioning progressed. Note: This series of figures is a simplified progression since it was found that stresses were high enough to cause delamination at the end of the detensioning sequence. Further refinement of the model is being developed for the retensioning which shows stresses through the wall section. No change is needed for the root cause report.

Comment 5 suggests a change in terminology from "volume" to "surface area". The change has been made.

Comment 6 suggests adding the de-tensioning of the 10 vertical tendons to the wording. The suggested clarification of the vertical tendon detensioning was added.

Comment 7 indicates the chronology should include which types of tendons were de-tensioned when. A conscious decision was made in the writing of the report to refrain from a detailed discussion of the specific tendon detension sequence. The actual de-tensioning sequence is available and discussed in the Failure Mode exhibit FM 7.3 Ex 1. No change required.

Misc Notes:

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Response By:	Charles Williams		
Reviewed By:		**Date Response Provided:**	8/9/2010
Status:	Closed	**Date Closed:**	8/11/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 176

Individual Contacted: Paul Fagan

Date Contacted: 7/21/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: PII Report:

On page 87 of PII report, Figure A2.5, there seems to be something inconsistent with the way the figure is plotted. There is sudden jump from azimuth 139 to azimuth 164 and azimuth 180 is not in the same straight line as azimuth 360.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

This is an anomaly in the laser scan data where there was no data taken due to obstructions. When the data was plotted it omitted the azimuth without data. The program used to plot this data skipped to the next available data. This plot is used to provide a comparison of the PII model vs laser scan data so the skipped section (where data was not available) does not hinder that comparison.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/4/2010

Status: Closed

Date Closed: 8/9/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 177

Individual Contacted: Paul Fagan

Date Contacted: 7/21/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: PII Report:

Refer to Exhibit 6, Figure 1 of both FM 1.2 and FM 1.15 in the PII Report. Please clarify what this contour plot represents. Item 1 under "Data to be Collected and Analyzed" on page 149 of the PII report states: Finite Element Modelling (FEM) of a local area around hoop tendons showing the stress levels in the concrete around the tendons sleeves due to tensioned vertical and hoop tendons (FM 1.2 Exhibit6). Why are the stress levels shown in this figure significantly different from those in Figures 3.4 and 3.5 in the main body of the PII report? Please confirm if the plots shows stresses after the concrete cracks at locations where stresses exceed the tensile capacity.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Revised Response:
The purpose of Failure Modes 1.2 and 1.15 was to evaluate the existence of stress concentrations in the area of the tendons. FM 1.2 Exhibit 6 is a visual representation of the stresses in the area of the horizontal tendons in between the vertical tendons. It was originally generated for the specific purpose of increasing the discrimination at low stresses to identify those areas below 200 psi which were well away from delamination stresses. The computer plotting routine cannot increase the color discrimination for low stresses unless the upper limit is cropped. For that reason the color scale was cropped at 200 psi. It was not intended to indicate that the maximum stress was 200 psi and the comment in the figure label indicated the threshold was 200 psi. The plot does not incorporate concrete cracking so the stresses identified in Figure 1 are those prior to any concrete cracking in response to the high stresses.

With the exception of the peak stress cut-off at 200 psi Figure 6 generally provides the same picture as Figure 3.5 of the Root Cause Report. For example, the peak stress midway in between the two hoop tendons is roughly 80 psi in both plots. The peak compressive stress at the 9 o'clock position is about -700 psi. The butterfly shape of the stress peak between the hoops is visible in both plots. The stresses shown in Fig 3.4 are higher since this is a location on horizontal tendons that is near the intersection with vertical tendons. No change required.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/4/2010

Status: Closed

Date Closed: 8/9/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 178

Individual Contacted: Paul Fagan

Date Contacted: 7/21/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: PII Report:

The PII report identified excessive radial tensile stresses in the plane of the hoop tendons (FM 1.2) and radial tensile stress concentrations around the hoop tendons sleeve holes (FM 1.15) without radial reinforcement as contributing failure modes to the CR3 delamination event. Refer to FM 1.2 on pages 149-150 and FM 1.15 on pages 167-168 of the PII report. The "Discussion" section of FM 1.2 states: "Note that vertical compression from the vertical tendons leads to an increase in radial and hoop tendon tension by Poisson's effect. As a simple approximation, the vertical compressive stress is approximately 1000 psi and the Poisson's ratio is approximately 0.2 so that the tensile stresses generated could be of the order of 200 psi. However, this is not an issue because:
1. In the radial direction, the concrete is constrained by hoop tendons so the vertical compression translates into hoop tendon load and does not generate radial tensile stresses;…"
The NRC SIT notes that this conclusion may not be true since it seems apparent (as explained later below) that the prestressing force in the vertical tendons has a very significant contribution to producing radial tensile stresses and stress concentrations in the vertical plane of the hoop tendons. This could result in small cracks in concrete at the tendon sleeve holes at the time of initial tensioning (as noted in FM 1.15) especially so if the bond between the sleeve and concrete is not strong.

FM 1.2 "Description" section identifies the origin of radial tensile stresses in the plane of the hoop tendons to three factors: Compressive force due to hoop tendons, stress concentration factors, and thermal effects. The NRC SIT notes that it does not identify the compressive force in vertical tendons as a factor, although it seems apparent (as explained later below) that the prestressing force in the vertical tendons has a significant contribution to producing radial tensile stresses and stress concentrations in the vertical plane of the hoop tendons.

The technical paper and typical industry calculation included as Exhibits 1 and 2 of both FM 1.2 and FM1.15 provide a common theoretical approach used for predicting/calculating the average radial tensile stresses in a cylindrical shell in the plane of the hoop tendons specifically due to prestressing forces in only the hoop tendons. The approach does not include the effect of prestressing forces in the vertical tendons in producing radial tensile stresses in the plant of the hoop tendons. The NRC SIT notes that this approach provides a good estimate of the average tensile stresses in the vertical plane of the hoop tendons due to prestressing forces in the hoop tendons but does not account for stress concentrations. For CR3, the average radial tensile stress at the time of initial tensioning (tendon force of 1633 k corresponding to 0.7Fu) was estimated using this approach to be of the order of 30 psi to 40 psi (Refer Exhibit 2 of FM 1.2 & FM 1.15, and Figure 3.3 of PII report). The stress concentration effects due to the hoop prestress is not expected to exceed 6 to 8 times the average tensile stresses, and therefore is not of a high enough magnitude to cause small cracks of the concrete at the interface of the tendon sleeves during initial tensioning.

The phenolphthalein testing of the specimen taken near the hoop tendon sleeve from the CR3 delaminated surface in FM 1.15 (Exhibit 8) observed pre-existing crack (~5mm long) at the intersection of the fracture surface and sleeves that could be attributed to the stress concentration effects around the tendon hole. Figures 3.4 and 3.5 on pages 21 & 22 of the PII report provide the stress contours of principal tensile stresses around the hoop tendon sleeves, based on uncracked linear elastic finite element analysis, due to combined effect of lock-off forces in the hoop and vertical tendons. Figure 3.4 reports peak tensile stresses as high as 1630 psi near the 12 o'clock (and 6 o'clock) locations of the hoop tendon sleeve hole. This well exceeds the tensile capacity of CR3 concrete and would cause small cracks. FM 1.15 discusses that it is likely that such non-active cracks would be present in other plants not analyzed here because the stresses calculated using linear elastic mechanics are very large and would exceed the tensile strength of all concrete materials. FM 1.15 also discusses that tendon sleeve hole locations and hoop planes generically possess higher potential for crack propagation, if exposed to additional drivers, due to this "weakening" of an already reduced effective concrete thickness between ducts. The NRC SIT notes that the effect of these peak tensile stresses around the tendon sleeve holes also depend on the bond between the tendons sleeve and the concrete.

FM 1.15 also highlights that standard industry analysis of this type (as in FM 1.15 Exhibits 1 & 2 that predict average radial tensile stresses due to hoop tendon forces only) has led to the mindset that large margins exist and a delamination as observed at CR3 would not have been predicted using industry standard calculation tools. The discussion on pages 4, 19, 73, 74 of the PII report generally focused on average tensile stresses due to only hoop tendons from typical industry analysis.

The NRC SIT notes that for CR3, the vertical compressive stress on the hoop tendon sleeve (in the plane of the hoops) due the lock-off force of 1633 k in the vertical tendon (accounting for bending due to the 6" eccentric location of the vertical tendon with respect to the centerline of the containment wall) is of the order of 1600 psi. The average vertical compressive stress due to lock-off force in the vertical tendon without considering the eccentricity of the vertical tendon is of the order of 1100 psi. Applying the closed form linear elastic solution of an infinite plate with a circular hole under uniform compression (Refer to FM 1.15 Exhibit 7), the compressive prestress in the containment wall due to the vertical tendon force at lock-off will cause a peak radial tensile stress between 1100 psi and 1600 psi (say ~1350 psi average estimate) in the vertical plane of the hoop tendons at the hoop tendon hole location. Thus, it seems apparent that over 1350 psi (i.e., over 80%) of the 1630 psi peak radial tensile stress at the hoop tendon hole location reported in Figure 3.4 of the PII report can be attributed as the contribution of the vertical tendon prestressing force in producing radial tensile stresses in the delamination plane of the hoop tendons. This is of a significantly high magnitude that can cause the concrete to crack at the tendon sleeve hole. The significantly high contribution of the vertical prestressing force in producing radial tension in the plane of the hoop tendons can also be observed qualitatively from calculation S10-0003 "Containment Repair Project – Conduit Local Stress Analysis."

The NRC SIT notes that it seems apparent that there is a mindset in industry that radial tensile stresses in the potential delamination plane of the hoop tendons is only caused by prestressing force in the hoop tendons and that prestressing force in the vertical tendons do not produce radial tension in the plane of the hoop tendons. Based on the above discussion and observations, it seems apparent that the prestressing force in the vertical tendons has a predominantly significant contribution (seems to be of the order of over 80%) in producing peak and average radial tensile stresses in the vertical plane (delamination plane) of the hoop tendons that can cause cracks at the tendon holes.

Please address the significance and contribution of the prestressing force in the vertical tendons in producing radial tension in the vertical plane (delamination plane) of the hoop tendons and potential crack at the tendon hole and its impact in the CR3 Containment Delamination Root Cause Report.

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

Revised Response:

This question requests additional analysis and discussion of the relative roles of vertical and horizontal tendons in the CR3 delamination. The analysis calculates actual conditions at a given location due to all the contributing factors whether they be from horizontal tendons, vertical tendons, the placement of rebar, or stress concentrating factors. The simulation of the delamination event was achieved based upon the entire set of conditions. It was not the intent to divide the event up into the quantitative assessment of each individual factor which would be entirely different at any other nuclear plant. We agree that the vertical tendon pre-stress contributes to the concrete radial tensile stresses. PII does not agree with the suggestion that the vertical tendons contribute over 80% of the radial tensile stresses.

Peak stresses at the horizontal tendon holes drop off very rapidly (a factor of 2 for every inch). The position of the crack is thus a very sensitive indication of the peak radial tensile stress. The purpose of the report is to draw overall root cause conclusions vs separating the effect of the horizontal vs vertical tendons.

The discussion section of FM 1.2 has been altered to acknowledge the contribution of the vertical tendons to the radial stress. Ex 6 was also replaced with figures from the PII Root Cause Report demonstrating this condition. The root cause report has also been revised to reflect the vertical tendon contribution.

Misc Notes:

Response By:	Charles Williams		
Reviewed By:		**Date Response Provided:**	8/12/2010
Status:	Closed	**Date Closed:**	8/12/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 179

Individual Contacted: Steve Cahill

Date Contacted: 7/27/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: Page 72 (3rd thru 5th paragraphs) of the PII report discusses modeling of tendons. It is stated in the 3rd paragraph that: "The tendons are assigned the cross-sectional stiffness of the sleeves and loads are applied as below. …"
(i) Please confirm whether or not the sleeves (conduits) for the hoop tendons and the vertical tendons were explicitly modeled in addition to the tendon holes in the concrete. If they were modeled, please explain how the conduit was modeled including the thickness and conditions at the contact boundary at the interface between the concrete hole and the tendon conduit. If not, why were they excluded?
(ii) Please confirm whether or not the vertical tendons were modeled explicitly as was done for the curved tendons.

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Revised Response:

One of the challenges of this project has been the simultaneous need to have very detailed modeling to accurately calculate the local stresses and to model the entire containment to capture stresses due to asymmetry in the overall building. PII's solution was to develop three models at different levels of dimension, detail, and complexity. The global model maps the entire containment to provide displacement information to the other models. The panel model maps each bay alone using input from the global model for dimensional boundary conditions. It then assesses the concrete and identifies cracking and damage if conditions dictate. It still has a relatively coarse mesh size, however, and so it is still somewhat simplified. The third model is a fine mesh model (about a 1 cubic inch element size) which is used to precisely analyze local conditions for the best characterization of actual concrete conditions. This model accurately portrays the sleeves with shell elements that share nodes with the surface of the holes in the concrete. The sleeves are modeled to be 1/8" thick. The hoop tendon loads are applied as a traction pressure on the inner half face of the conduit surfaces. The vertical conduits are modeled in the same way geometrically, however the reaction force of the vertical tendons is only applied to the top of the ring girder in the global model, and this load is transferred to the submodels using mapped displacements from the global model. The concrete elements are assumed to be bonded to the sleeve elements. The bonding criteria used allows the concrete to crack around the tendon sleeves as expected. The early version of the crack propagation model for the root cause included the pre-existing cracks. The current version of the model being used for retensioning does not include pre-existing cracks. The current model shows that it did not need to have pre-existing cracks as a pre-existing condition to have the delamination event during the SGR detensioning. When a crack does develop the computer model keeps track of the positions of the broken elements. If compressed, the crack closes and passes the stress which prevents one element from penetrating the location of another node.

Misc Notes:

Response By: Charles Williams

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Reviewed By:		**Date Response Provided:**	8/12/2010
Status:	Closed	**Date Closed:**	8/12/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 180

Individual Contacted: Steve Cahill **Date Contacted:** 7/27/2010

Requestor/Inspector: Louis Lake **Category:** Question

Request: On page 202 of the PII report, FM 4.5 concludes that a likely number for the creep coefficient today is 1.63 and that creep did not generate the delamination. Based on data of recent CR3 creep testing reported in Exhibit 14 of FM 4.5, the ultimate creep coefficient for CR3 concrete is reported to be 2.39 on page 7 of FM 4.5 - Exhibit 15. Further equation (21) is provided on page 7 of FM 4.5 - Exhibit 15 for determining the creep coefficient for any other time. Using equation (21) in FM 4.5 - Exhibit 15, the creep coefficient for CR3 concrete at the time of SGR outage (~33 to 36 years from the time of initial tensioning) would be estimated to be around 2.39. However, the fracture-based Abaqus model used in the root cause analysis used a creep coefficient value of 2.2. Please reconcile the 3 different values of creep coefficient (1.63, 2.39 and 2.2) reported in the root cause report for the existing CR3 containment concrete at the time of the SGR outage, and provide the justification/basis for the value of 2.2 used in the computer simulation. Please explain why creep is not a contributor to the delamination when the creep coefficient is a significant input parameter in the visco-elastic fracture-based computer model used to simulate the delamination?

References:

Response Assigned to: Charles Williams **Date Due to Inspector:**

Response:

Revised Response:

The creep ratio is an analysis input but it is a difficult parameter to actually measure outside of a laboratory environment. FM 4.5 Exhibit 3 was an estimate of the creep ratio based upon generic adjustment factors and a creep ratio of 1.63 was obtained.

PII also pursued temperature accelerated creep tests at the University of Colorado. That testing obtained a creep ratio estimate of 2.39.

Creep is a phenomenon whereby concrete under constant stress will exhibit increasing strain over time. It is affected by at least four independent parameters:
- o Time under sustained stress
- o Time cured prior to applying sustained stress
- o Temperature to which the concrete is exposed
- o The specific materials in the particular concrete sample

Fortunately the four parameters appear to act independently so that the total strain ratio is the simple product of four functions, each one dependent only on the one variable alone. Thus it is possible to estimate the creep coefficient at CR3 during the SGR outage by adjusting sample data for specific difference. In the case of the University of Colorado tests, the samples were from CR3. They were heated to either 80 C or 40 C to accelerate the creep process. They were under sustained compression for about two weeks rather than 33 years in the containment. They cured for 36 years prior to being compressed (in the test) rather than for 2.5 years in the plant. Thus corrections to the data were needed in three of the four categories.

CR3 performed two creep tests in 1970. The test was at ambient temperatures. The cure time was about 90 days rather than 2.5 years, and the test ran out to 6 months rather than 33 years. Adjusting for the longer period of service, a result of about 2.45 for the creep ratio is obtained. Other data provide similar, but slightly different, results. PII determined that a creep ratio of 2.2 was a good estimate of the true value.

Creep generally reduces stresses by the relaxation associated with the long term increase in strain. However, if the force causing the stress is removed, the concrete sees the immediate change in strain associated with the elastic strain which is followed by additional change (known as creep rebound). If concrete is immediately tensioned and then de-tensioned it ends up without stress in its original position. If concrete is tensioned for a long time and it is then de-tensioned it does not return to its original stress-free condition because it crept in the interim. As it seeks the new stress-free condition the steel components in the concrete seek to return to their original stress-free condition and, in the end, a compromise occurs which leaves the concrete partially stressed. It is in this way that the creep coefficient can impact the stresses seen in a de-tensioning activity.

PII performed its analyses at a creep ratio of 2.2. However, the case was also run with a creep ratio of 1.5. The peak stresses were reduced by about 15 to 20 percent. PII's assessment was that the delamination event would have occurred with creep ratio within the estimate range.

There are many inputs to the computer model. Any one of them (elasticity, fracture energy, tensile capacity, et cetera) might be identified as a contributor to the delamination on that basis but the process used is to look for abnormal conditions which played an extraordinary part in causing the event. As indicated above, the event is not highly sensitive to the value of the creep ratio. Standard materials show similar creep ratios.

ASTM C 512 provides creep data which can be adjusted to CR3 conditions and the creep ratio obtained is 2.14. The concrete material was identified as a contributor on the basis of its unusual and significant properties. Creep was not identified because it was not unusual for the service condition.

Misc Notes:

Response By:	Charles Williams
Reviewed By:	
Date Response Provided:	8/9/2010
Status:	Closed
Date Closed:	8/11/2010

Request Number: 181

Individual Contacted: Steve Cahill

Date Contacted: 7/27/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: Why is it that the fracture energy and tensile strength thresholds for delamination used in the delamination simulation (Gf = 0.08 lbf/in and f't = 108 psi) different from that used (Gf = 0.14 and f't = 200 psi) in the margin-to-delamination analysis for expanded detensioning?

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

Revised Response:

As discussed in response to NRC SIT Request 179, the modeling of this event is a challenge based upon the finite amount of computer capacity available to run the model. The solution was to develop three models each doing part of the job. The most exact result comes from the detailed model. It uses a very fine mesh size and benchmarks well with measured parameters such as Gf , F't, and Eo. The panel model and global model suffer from a lack of detail due to the larger mesh size required by the larger structures being modeled. As a result they fail to fully resolve the stress concentrations near the tendons. This can be addressed by inserting artificial stress concentration factors or equivalently by changing the input parameters to respond to the lower stresses calculated by the coarse mesh. This is referred to as calibration. The margin-to-delamination analysis preceded the development of the detailed model so its inputs were calibrated as indicated above.

The delamination simulation has to be run on the global model or the panel model so they also use calibration input values.

A common practice in finite element modeling is calibration. It consists of adjusting material property input assumptions as needed to reproduce the benchmarked performance being simulated. For example, if the computer predictions over-estimate the radial displacements then the assumed value of elasticity may be increased appropriately to result in a fit for radial displacements. Similarly, if the stresses calculated by the model are too low to predict cracking when cracking is observed, then tensile capacity and fracture energy can be reduced to the point where the code properly predicts cracking. This was the approach used in the margin-to delamination calculations. Analysis of the situation identified the under-estimation of stresses was the result of the coarse mesh spacing used by the model. It was not possible to decrease mesh spacing due to computer capacity limitations so a new sub-model was developed with fine mesh spacing. This change led to agreement between the model and benchmarks using actual measured material properties.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/9/2010

Status: Closed

Date Closed: 8/11/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 182

Individual Contacted: Steve Cahill

Date Contacted: 7/27/2010

Requestor/Inspector: Louis Lake

Category: Question

Request: General Comment on Recommendations/Corrective Action to Prevent Recurrence (page 67-68 of PII report and page 3 of CAP Report): Based on the two CR3 delamination events (dome and cylinder), an important corrective action to prevent or minimize the potential for structural damage (including delamination and other) to the containment during major modifications would be to "recognize" upfront the potential technical issues and adverse effects of certain important "inherent characteristics" of the CR3 containment structure, specifically those identified by FMs 3.4 (fragility of the coarse aggregate), 2.12 (lower than normal tensile strength) 1.2 (high radial tension with no radial reinforcement), 1.15 (stress concentrations around tendon holes), 1.1 (high prestress level), and significantly higher creep in the existing concrete (FM 4.5), and explicitly factor them into the engineering and execution of possible future major modifications to the containment structure. Why is this not specifically emphasized in the corrective action?

References:

Response Assigned to: Charles Williams

Date Due to Inspector:

Response:

The Corrective Actions to Prevent Recurrence (CAPR) in both the PII report and the Corrective Action Program (CAP) investigation document identify the need to use the CR3 delamination experience when performing analysis for the upcoming retensioning and for detensioning more than one tendon/group in the future. It is understood that the CR3 delamination experience includes all contributors to delamination. The CAP investigation also specifically requires the use of the modeling methodology improvements for these evolutions.

Misc Notes:

Response By: Charles Williams

Reviewed By:

Date Response Provided: 8/4/2010

Status: Closed

Date Closed: 8/11/2010

Request Number: 183

Individual Contacted: Paul Fagan

Date Contacted: 8/10/2010

Requestor/Inspector: Frahad Farzad

Category: Question

Request: Calculation S10-0030 rev. 0

In section 6.3.2.1 where out of plane shear check is performed for service load combination, the yield stress of rebar "40,000 psi" is used to determine the required area of stirrups. This is not consistence with working stress design methodology of ACI 318-63 and is outside the CR3 design basis requirements. Please explain.

Follow-up:

In the paragraph discussing axial compression at elevation 245, validate that the correct units have been used for axial compression and M'.

Ensure that Revision 32 of S10-0030 incorporates the information discussed in this response.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

(NOTE: the two follow-up items are inserted into the original response)

During the development of S10-0030, the existing stirrups at elevation 245 above the repair area were checked for out of plane shear. In determining the allowable stress limit for these stirrups, ACI 318-63 section 1003 was reviewed. Because stirrup reinforcement was not specifically identified in Section 1003, the stress limit of 40,000 psi for spiral reinforcement was used because the stirrup function was determined to be similar to spiral reinforcement. After further review, the stress limit of 20,000 psi specified in Section 1003(a) for all other reinforcement would be the appropriate limit to use.

Because the stress limit for the existing stirrups is only 20,000 psi, an alternate method to accept the existing stirrups for out of plane shear is being evaluated as described below.

The CR3 design basis for the containment building employs a combination of ACI 318-63 working stress and ultimate strength design methods. Section 5.2.3.3.2 of the FSAR specifies use of the ultimate strength method for design of shear reinforcing. In the rebar design calculation S10-0030, Section 6.3.2.1, it was recognized that the highest calculated shear forces occurred in load combination 11, which is a working stress load combination. Also, these shear forces occur at a cross-section outside (above) the SGR repair area. The shear stress evaluation in this area was provided as an additional check, in addition to those evaluations specifically required by the CR3 FSAR for the delamination repair area. As such, the evaluation was performed as a working stress evaluation.

Alternatively, the analysis could credit the increased shear capacity in the cross-section due to axial loads, which is allowed per ACI 318-63, Section 1201(e). At present, this added capacity was conservatively omitted in the calculation.

ACI 318-63 does not clearly state the application of Equation 12-2 and 12-4; however, ACI 318-08 commentary states the following (paraphrased): for increasing axial compression values, the M' value from Eq. 12-3 decreases toward zero and then becomes negative. For negative values of M', the value of Vc obtained from Equation 12-2 has no physical significance and the value provided by Equation 12-4 for Vc should be used instead.

For the case mentioned in the question above, i.e. Calculation S10-0030, Revision 2, Appendix H, Load Case 11,the axial compression at Elevation 245.6' and for Cut #1 is equal to 509.89 kips/ft, the bending moment = 51.61 k-ft/ft, t = 42" and d = 37.635". The associated value for M' is:

M' = M - N (4t-d)/8

= 51.61 k-ft/ft – 509.89 k/ft (4 x 42" – 37.635")/(8 x 12) = -640 k-ft/ft

(Note: divide by 12 to convert inches to feet)

In this case, Vc should be calculated according to Equation 12-4 as follows:

(see request folder for calculation)

Since the shear capacity of the concrete section exceeds the shear demand in the section, shear reinforcement is not required. The S10-0030 calculation will be revised to credit the concrete capacity Vc in cases of high axial compression. Note: Calculation S10-0030, Revision 2 includes the necessary changes that credit concrete capacity in areas of high axial compression.

Reference:

Reference 1: 32-9131877 Revision 2: Crystal River 3 Containment Shell Design for the Construction Opening and Delaminated Areas
(see request folder for reference)

Reference 2: S10-0030 Revision 2: Reinforcement Design for Containment Delaminated Wall (Not Approved)

NOTE: Reference 1 will be included as an attachment to Reference 2

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/27/2010

Status: Closed

Date Closed: 11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number: 184

Individual Contacted: Paul Fagan

Date Contacted: 8/10/2010

Requestor/Inspector: Frahad Farzad

Category: Question

Request: Calculation S10-0030 rev. 0

Based on spot check of appendix I, "radial shear design in the hoop direction" there are extremely large numbers calculated for Vci for a number of cuts. For example, on page 915 of attachment A, for cut number 12, 66,417.68 is entered under the column Vci where as for cut number 11 "with similar parameters to cut number 12", 42.26 is entered. Considering the lesser of Vci and Vcw is used as shear allowable, this apparent inconsistency in value of Vci for cut number 12 will effect the results. Please address the effects of this inconsistency on the overall results and whether there is any impact on the adequacy of the containment repair design. In addition provide confirmation that all tables and numerical calculations in calculation S10-0030 have been verified in accordance with your quality assurance program.

Follow-up request:

1.Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall (Reference 1), Page 53 calculates the value of d = 26.625". When this value is substituted into the equation for calculating Vci shown on Page 52, the resulting Vci is considerably less than that calculated in Appendix I, Load Case 4, Page 916, cut number 12, where Vci = 66,417.68 kips. Please explain this deficiency.

2.Calculation S10-0030, Appendix I, Load Case 4, Page 916, cut number 11, has very similar input values for Vu, Mu, Mcr, etc., as cut #12, however the reported Vci (in the 9th column; Vci when N>100 psi) = 42.26 kips, whereas for cut 12, as noted in Item 1, this value is 66,417.68. There appears to be an error in the Excel spreadsheet. It would appear, based on the input values, that these two results should be similar. Please explain this deficiency.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Please refer to Equation 1 on Page 51 of the calc (Equation 26-12 of the ACI 318-63); the second term in the equation has (M/V - d/2) in the denominator. For this particular section cut, the calculated value is a small number and because it appears in the denominator, the result for Vci becomes a large value. The Code specifies use of the smaller value calculated via equation 26-12 and 26-13. In this case, Equation 26-13 controls and is used for the concrete shear capacity. Therefore, the correct value for concrete shear capacity was used and the conclusion of the calculation remains valid. The disposition of the high shear value will be clarified in a revision to S10-0030.

All tables and numerical calculations in S10-0030 have been verified consistent with the vendor's quality assurance program.

Response to Follow-up Request #1 (11/8/10):
The results contained in Appendix I are for the radial shear in the hoop direction as shown below:

(see request folder for diagram)

The reason these results are for the hoop direction is that the axial compression/tension component must be perpendicular to the shear load plane.

Since the radial shear is acting across the vertical plane the "d" value is the distance from the extreme compression fiber to the centroid of the prestressing force, i.e. centroid of the hoop tendons, therefore d = 32" as shown on page 93, Case 2. The value shown on Page 53 of Reference 1 is only applicable to the radial shear on the horizontal plane in the vertical direction, the results for which are contained in Appendix H.

Referring to Calculation S10-0030, Appendix I, Page 916, Load Case 4, cut 12 at arc length 24.9 the following results are listed:

Vu = 2.42 k/ft
Mu = 3.23 kip.ft/ft
Mcr = 174.33 kip.ft/ft
ΦVci = 66,417.68 kips
ΦVcw = 149.39 kips
ΦVc = 149.39 kips
f'c = 5800 psi
fpc = 564.50 psi
b = 12", d' = 32", Φ = 0.85

Per FSAR Section 5.2.3.3.1 and 5.2.3.3.2 and ACI 318-63, Section 2610 the shear capacity is calculated as follows if the applied membrane compression is > 100 psi:

Section 2610(b) - The shear, Vci, at diagonal cracking shall be taken as the lesser of Vci and Vcw as determined by Eq. 26-12 and 26-13.

Vci = 0.6bd'(f'c)0.5 + Mcr / {(M/V – d/2)} + Vd Equation 26-12

(Vd is assumed to equal 0)

But not less than 1.7bd(f'c)0.5

And

Vcw = bd (3.5(f'c)0.5 + 0.3fpc) + Vp Equation 26-13

Making the substitutions into Eq. 26-12:

ΦVci = 0.85 {0.6x12x32x(5800)0.5/1000 + 174.33/[3.234562/2.421868 – 32/2/12]}

= 0.85 {17.547 + 174.33/0.0022316}

= 66416 kips ~ 66417 kips (Difference due to only using 6 decimal places)

Note: The values in the highlighted denominator (M/V – d/2) have been taken directly from the Excel spreadsheet out to 6 decimal places. If the values listed at the top of this page are substituted into Equation 26-12 then Vci = 107,626. Obviously, the equation becomes unstable for very small values of (M/V – d/2), however, since the Code states that the lesser value of Vci and Vcw as determined by Eq. 26-12 and 26-13c controls design, the number of decimal places used has no material effect on the final value of Vci.

As previously stated, the shear, Vci, at diagonal cracking shall be taken as the lesser of Vci and Vcw as determined by Eq. 26-12 and 26-13. For the case where Eq. 26-12 gives extremely high values for ΦVci due to the highlighted denominator having an extremely small value, then Eq. 26-13 will control since it will give a lower value. Note that when applying Eq. 26-13, the effective depth, d, shall be taken as the distance from the extreme compression fiber to the centroid of the prestressing tendons, or as 80% of the overall depth of the member, whichever is greater.

d = 32” or 0.8 x 42” □ d = 33.6” (applicable to Eq. 26-13 only)

ΦVcw =Φ bd (3.5(f’c)0.5 + 0.3fpc) + Vp Equation 26-13 (Vp conservatively taken as 0)

= 0.85 {12 x 33.6 x(3.5 x (5800)0.5 + 0.3 x 564.50) /1000}

=149.39 kips ~ 149.39 kips reported

Response to Follow-up Request #2:

Results from Appendix I - Load Case 4: 0.95D + Fa + Ex’ + Ev’ + P + Ta Cut #11 at 24.9’

Vu = 2.21 k/ft
Mu = 2.76 kip.ft/ft
Mcr = 174.58 kip.ft/ft
ΦVci = 42.26 kips
ΦVcw = 149.55 kips
ΦVc = 42.26 kips
f’c = 5800 psi
b = 12”, d’ = 32”, Φ = 0.85

Substitute above values into Eq. 26-12
Checks

ΦVci = 0.85 {0.6bd’(f’c)0.5 + Mcr / {(M/V – d/2)} + Vd} Equation 26-12

= 0.85 (17.547 + 174.58/[2.76/2.21 -32/2/12])

= 0.85 (17.547 + 174.58/-0.084)

= 0.85(17.547 – 2078) But not less than 1.7bd(f’c)0.5

= - 1752 kips < 0.85{1.7bd(f’c)0.5 }= 42.26 kips

Conclusion:
The spreadsheet results are correct.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/27/2010

Status: Closed

Date Closed: 11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:38 AM

Request Number:	185		
Individual Contacted:	Charles Williams	**Date Contacted:**	8/13/2010
Requestor/Inspector:	George Thomas	**Category:**	Information Request

Request: Please forward an electronic copy of procedures CAP-NGGC-0205 (Revision 11) and CAP-NGGC-0200 (latest revision).

References:

Response Assigned to:	Charles Williams	**Date Due to Inspector:**	

Response:

See response folder for requested procedures. Note that Revision 11 is not the latest revision of CAP-NGGC-0205. Revision 11 was used for the containment root cause. The latest revision (Rev. 12) was just issued this week (8/9/10).

Misc Notes:

Response By:	Williams / Miller		
Reviewed By:		**Date Response Provided:**	8/13/2010
Status:	Closed	**Date Closed:**	8/17/2010

Request Number: 186

Individual Contacted: Steve Cahill

Date Contacted: 8/17/2010

Requestor/Inspector: Farhad Farzam

Category: Information Request

Request:

Section 6.5 (Page 122) of Calculation S10-0030, Revision 0 states that rebar lap splices are provided based on 36 times the diameter of the bar. This criterion is the minimum lap splice length required according to Section 805 of ACI 318-63. Provide further information to show compliance with the following provisions stated in Section 805 (b) of ACI 318-63:

•The splice shall transfer the entire computed stress from bar to bar without exceeding three-fourths of the permissible bond values given in the code.

•Where more than one-half of the bars are spliced within a length of 40 bar diameters or where splices are made at points of maximum stress, special precautions shall be taken, such as increased length of lap and the use of spirals or closely-spaced stirrups around and for the length of the splice.

Follow-up request #1:

Clarify why the bond stress calculated on Page 56 of Calculation S10-0030, is based on (6.7 √f'c)/D whereas the Response to Question 186 stated that lap splices are based on providing bond stress equal to the minimum of:

(9.5 √f'c)/D nor 800 psi ACI 318-63, Section 1801©

Follow-up request #2:

The second bullet in Question #186 requests that further information be provided to show compliance with the following provisions stated in Section 805(b) of ACI 318-63:

• Where more than one-half of the bars are spliced within a length of 40 bar diameters or where splices are made at points of maximum stress, special consideration shall be taken.…

Further information is required showing compliance with the above requirements.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

With regard to the first bullet, ¾ of the minimum permissible bond strength for the lap splice, as provided by Section 1801(c), exceeds the capacity of the reinforcing steel (conservatively assuming that the stress in the bar is equal to the yield stress). Therefore, the splice length is adequate for bond development as shown below:

The length of the #11 lap splice provided is 61” (Ref. Drawing #421-358).

Bond stress as provided in Section 1801C is equal to the minimum of:

(see response folder for calculation)

¾ of the lap splice bond strength capacity = 0.75 x 0.564 x (π x D) x 61” = 114.2 kips, (where lap splice length = 36 x (D) 1.41” x 1.20 = 60.9”).

11 bar maximum tension strength capacity = 60 ksi x Ab = 94 kips < 114.2 kips OK

Where bar area Ab = 1.56 in2

With regard to bullet 2, the lap splice length was increased by 20% (Refer to Section 6.5 of Calc. S10-0030) to account for closely spaced lap splices in order to avoid splitting in the plane of the bars as provided in Section 805(b). This is consistent with and meets the intent of ACI Section 805b as detailed in the commentary of ACI 318-63.

Response to follow-up request #1 (11/10/10):

ACI 318-63, Section 1801(c)(1) states:

For tension bars with sizes and deformations conforming to ASTM A305:

Top Bars*	bond stress = 6.7√f’c/D nor 560 psi
Bars other than top bars	bond stress = 9.5√f’c/D nor 800 psi

*The code states: Top bars, in reference to bond, are horizontal bars so placed that more than 12” of concrete is cast in the member below the bar.

Calculation S10-0030 conservatively assumed the bond stress for top bars, i.e. 6.7√f’c/D when checking the required development length for the #5 radial ties on page 56. For hooked bars, ACI 318-2008 indicates that there is no distinction between top bars and other than top bars in the commentary section R12.5.

The bond stress for lap splices in the main vertical and hoop direction reinforcement was based on the Code requirement for “bars other than top bars”, i.e. 9.5√f’c/D.

As a basis for the definition of a top bar, in reference to bond, the description contained in Reinforced Concrete Design by Schaums Publishing Co., 1966 is reproduced below:

(see response folder for description)

The horizontal bars used in the repair design of CR3 containment wall in Bay 3-4 are not considered top bars due to the following observations:

- The mockup of the containment wall was cut through its entire height through the rebar and no evidence of voiding around rebar was found, additionally it was not possible to identify where the lift lines were when looking at the cut cross section. From this observation it can be concluded that no subsidence of the concrete away from the rebar occurred.

- The concrete sets very rapidly - time of set was measured at about 2 hours. This is primarily due to the very low water to cement ratio of 0.375 (S&ME Test Report for Phase III Testing).

- The Advacast (super plasticizer) performed very well producing a low viscosity, cohesive, self consolidating mix at very low water cement ratio.

- The lifts exhibited no water bleeding at the top of the five foot placements which is indicative that there was no excess water available to migrate upwards through the concrete.

• The concrete testing vendor, S&ME estimated the slump of the concrete mix without the addition of a super plasticizer (high range water reducer) at less than 1", which supports the conclusion that there was very little water available to the mix and that subsidence, if properly vibrated, would not be a problem.

• Concrete Specification for the SGR concrete, CR3-C-0003 required thorough consolidation of the concrete through the use of hand held vibrators. The concrete was placed in lifts 12-18" thick. Each lift was vibrated in order to consolidate the concrete before proceeding to the next lift. After placing the second lift, the vibrator was extended (according to the placement requirements) down inside the concrete of the previous lift in order to ensure consolidation of the concrete between the two lifts. This requirement ensured re-consolidation of the concrete on the top of the previous lift and eliminated the potential for trapped water in any top bar in the top area of the previous lift.

• The concrete mix used has been tested and shown to have very low shrinkage (Concrete Specification CR3-C-0003) which further eliminates the potential for a weaker bond around the reinforcement bars.

By reviewing the historical application and the background of the top bars provisions, it was determined that this provision is not applicable for hoop reinforcement bars in the CR3 containment repair design – Bay 34. However, the application of the top bar provisions on the splices of the horizontal bars shows that the splice length remains adequate. The following section further validates this statement:

For the horizontal bars, lap splices are used in two locations:

1-At Buttresses 3 and 4 for lap splicing #8 bars spaced at 12" c/c

The #8 hoop bars extending out from both buttresses have 40 ksi yield strength. The length of the #8 lap splice used in the design is 44" (Ref. Drawing #421-358).

Bond stress as provided in Section 1801-C and using the top bars provisions is equal to the minimum of:

(see response folder for equation 1)

¾ of the lap splice bond strength capacity = 0.75 x 0.560 x (π x D) x 44" = 58 kips, (where lap splice provided length = 36 x (D) x 1.20 = 44").

8 bar maximum tension strength capacity = 40 ksi x Ab = 40 x 0.79 = 31.6 kips < 58 kips OK

(Where # 8 cross section area Ab = 0.79 in2)

Notice that ACI 318-63 Section 805 requires the computed stress to be used but the bar yield stress was used here conservatively for comparison with the lap splice bond capacity.

2-Above the equipment hatch between elevation 165' and 176' and at arc length zero for lap splicing #10 bars spaced at 12" c/c

In this area of the containment wall in Bay 34, #10 bars are staggered with #11 bars at 6" spacing. Per Calculation S10-0030, Appendix E, the maximum bending moment in combination with axial tension (in the area of interest between elevation 165' and 176') occurs at Cut #20 at arc length 0.0' and are:

M = -50.41k-ft/ft

N = 92.58k/ft (tension) for Load Case #1).

Compute the actual stress in the outer layer of reinforcement in order to evaluate the bond strength requirements. In addition to the outer layer of reinforcement (Area = 2.83 in2/ft), there is an inner mat consisting of #11s @ 12”c/c (Area = 1.56 in2/ft). The total reinforcement available to resist axial (membrane) tension is 2.83 + 1.56 = 4.39 in2/ft. The resulting computed tensile stress in the #10s due to the axial tension can be determined as follows:

fs = (92.58 kips) / (4.39 in2/ft) = 21.08 ksi

The resulting tensile stress in the #10s due to bending stresses can be determined as follows:

Moment from Load Case 1 = -50.42 k-ft/ft (Compressive stress at the outer face)

Moment from accident thermal gradient = 111 k-ft/ft (See Section 6.1 in S10-0030)

Resulting moment in the section Mu = 111 – 50.42 = 60.58 k-ft/ft

ΦMn= ΦAs x fs x j d

As= 2.83 in2/ft, d = 39.045”, ΦMn = 60.58 k-ft/ft

Assume j = 0.9 → 60.58 x 12 =0.9 x 2.83 x fs x 0.9 x 39.045

→ fs = 8122 psi = 8.1 ksi
Total tensile stress on the outer layer of rebar = 21.08 + 8.1 = 29.2 ksi

The lap spliced #10 hoop bars between elevation 165’ and 176’ have 60 ksi yield strength. The length of the #10 lap splice used in the design is 55” (Ref. Drawing #421-358).

The allowable bond stress as provided in Section 1801-C and using the top bars provisions is equal to the minimum of:

(see request folder for equation 2)

¾ of the lap splice bond strength capacity = 0.75 x 0.441 x (π x D) x 55” = 72.6 kips, (where lap splice provided length = 36 x (D) x 1.20 = 55”).

10 bar computed tension demand = fs x Ab = 29.2 x 1.27 = 37 kips < 72.6 kips OK

Where # 10 cross section area Ab = 1.27 in2

In conclusion, the provisions for the top bar splices are not applicable on the lap splices of the horizontal bars in the containment wall. However, the splice length provided on Drawing 421-358 is also adequate if these bars are conservatively considered to be top bars.

Response to follow-up request #2 (11/10/10):

As was previously noted in the original question #186 response, all lap splices were increased by 20% to account for lateral spacing of splices less than 12 bar diameters (bd) or closer than 6” or 6 bar diameters from an outside edge. This increase in length also addresses the concern where more than one-half of the bars are spliced within a length of 40 bar diameters. The ACI 318-63 Commentary for Section 805 (splices in reinforcement) states:

Splices should, if possible, be located away from points of maximum tensile stress. The Code requires special precautions at points of maximum stress or when more than half the bars are spliced within a 40 bar diameter length. Such splices should have both extra length and extra ties or spirals if the splices are spaced laterally closer than 12 bar diameters; for more widely spaced splices either extra length or extra ties or spirals are adequate.

When the lateral spacing between lap splices is less than 12 bar diameters or the splice is located closer than 6 bar diameters or 6 in. to an edge, the lap must be increased an additional 20 percent,

Based on the above Commentary, the 20% increase in lap splice addresses both issues, i.e. closely spaced bars and maximum tension. Also note that per Drawing 421-358 the total distance between adjacent splices = splice length + 30", which for a #10 bar = 55" + 30" = 85" = 67bd, and for a #11 bar = 61" + 30" = 91" = 65bd, both exceeding the 40bd criteria stipulated in the bulleted item above, therefore, only the second provision applies (max. stress).

The ACI 318-63 Code was written primarily for normal building construction, i.e. beams and columns and the reference to "points of maximum stress" refers to the point of maximum bending moment/tensile stress in a beam or column. In Bay 34 there is no specific point of maximum bending moment, the moments vary considerably from positive to negative bending across the width and height of the wall. Generally, in Bay 34 lap splices were located in areas of relatively low tensile stress (membrane + bending being the primary stress consideration). For example, the membrane + bending stresses for the lap splices in the hoop direction (outside face) between Elevation 165' to 176' range from 12.22 psi tension to 38.7 psi compression (See Calculation S10-0030, Appendix C, Cut #20 from arc length 0.0 to 9.00). The hoop splices between Elevation 218'-6" and 237'-9" adjacent to the buttress, consist of new #11s lap spliced to existing #8s. The membrane + bending tensile stress in this region is 397 psi (Calculation S10-0030, Appendix C, Cut #3 arc length 24.9'), and the lap splice length should be 24 bar diameters based on the existing #8 being 40ksi steel. However, Calculation S10-0030 required a splice length of 1.2 x 36 bar diameters = 43 bar diameters, which is 180% of the basic splice length. Vertical lap splices are also generally located in areas of relatively low tensile stress, except for the splices located in the outer face at Elevation180'. The tensile stress in this region is about 650 psi. However, in this area an inner mat of #11s has been provided which is tied to the #11s on the outer face with #5 ties, basically forming vertical column cages. The splices at this location have therefore been provided with both extra length (20%) and lateral ties. Notice that all vertical bars are on the inner side of the hoop bars. In summary, special consideration described in the code has been applied.

The following additional as-built details should be noted:

- In the hoop direction #10s were lap spliced on the outside face from Elevation 165' to 176' and staggered at approximately 51", therefore, adjacent splices are spliced within a length = 51" +51" = 102" = 80db > 40 db (Attachment #1 for as-built drawing).
- In the hoop direction, in the mid-layer, #11s were lap spliced on the outside face from Elevation 176' to 201'. These splices were staggered approximately 12' (Attachment 2 for as-built drawing) which far exceeds the 40 db provision of the code.
- In the hoop direction #11s were lap spliced with existing #8s at the buttresses. Since the existing bars are 40 ksi, the splice length required by ACI 318-63 is 24 bar diameters. The actual minimum splice length provided was 1.2 x 36 bar diameters (Refer to drawing 421-355 and Calculation S10-0030, Section 6.5). Note that the 20% increase in length is applicable to both the 24 and 36 bar diameters. Therefore total splice length provided = 36 x 1.2 = 43.2 bd > 24 x 1.2 = 28.8 bd required by the code.
- In the vertical direction, the hoop reinforcement provides confinement to the vertical bars. Additionally, lateral ties were provided (Refer to drawing 421-359 in all the vertical rebar cages, thus meeting the code provision for both extra length (20%) and lateral ties.

All lap splice lengths were increased a minimum of 20% per ACI 318-63, Section 805(b)

Summary:
All lap splice lengths were increased by 20% above the code required minimum splice length of 36 db for 60,000 ksi reinforcement. This 20% increase in splice length addresses both issues of closely spaced bars and maximum stress. Although there is no specific point of maximum bending moment/stress in the wall, unlike a

beam or column, the majority of the splices were located in areas of relatively low tensile stress. The only exception to this rule was the vertical splices located in the outer face at Elevation 180'. However, these splices are confined by lateral ties and, as previously noted, increased in length by 20%. Based on this assessment, all splices are acceptable and meet the ACI 318-63 code requirements.

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	9/7/2010
Status:	Closed	**Date Closed:**	12/17/2010

Request Number: 187

Individual Contacted: Steve Cahill

Date Contacted: 8/17/2010

Requestor/Inspector: Farhad Farzam

Category: Information Request

Request: The original design of CR-3 provided sufficient prestressing that there was no need for reinforcing steel to resist membrane tension in the area of the wall under repair in Bay 3 4 (Figure 5-12 of the CR-3 FSAR). A biaxial tension condition in Bay 3-4 is predicted as shown in tables included in Attachment A to Calculation S10-0030. Provision of CC-3532 of ASME Section III, Division 2 requires positive mechanical splice or welded splice in a region where tension is predicted perpendicular to the bar to be spliced, unless calculations or tests of the selected splice detail are made to demonstrate that there is an adequate transfer of force. Considering Item 1 above and as the original code of record, ACI 318 63, does not address splicing reinforcing steel in an area where biaxial tension is predicted, lap splicing the new reinforcing steel in the repair area (e.g., as shown in Figure 1-2 of Calculation S10-0030) where biaxial tension is predicted, is contrary to the current industry code requirement and does not provide assurance for splice integrity. Please explain and provide further information relative to the acceptability of lap splice in the area where biaxial tension is predicted.

Follow-up request:

For Area 2 explain why it is acceptable to exceed the acceptance level of 251psi for stress normal to the lap splice that has been established in the Response.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Updated response (11/10/10):

Code of Record

ASME Section XI IWL-4230 (Repair and Replacement activities for reinforcing steel), states that the original construction code applies for repair activities. The code of record for the CR3 concrete containment, ACI 318-63 (Ref. FSAR 5.2.3.1), does not require mechanical splices in areas where biaxial tension exists. Related ACI provisions are included and have been implemented.

Review of ACI 318

ACI 318-63, Section 805 specifies that: lapped splices in tension shall not be used for bar sizes larger than #11 and that splices at points of maximum tensile stress shall be avoided wherever possible; such splices where used shall be welded, lapped, or otherwise fully developed. These requirements were incorporated into the design and specified and followed in project design documents. Considering the use of lap splices in the original design at dis-continuities, i.e. the equipment hatch, and the code of record requirements, lap splices were installed during original construction and are acceptable for bar sizes #11 and smaller in areas with biaxial tension.

Later editions of ACI 318 also do not require mechanical splices in areas where biaxial tension exists. ACI 318-05 Chapter 12—“Development and Splices of Reinforcement” is silent on biaxial tension situations. ACI 318-05, Chapter 19—“Shells and Folded Plate Members” discusses reinforcement to resist both membrane and flexural stresses and makes clear that tensile reinforcement shall be provided in two or more directions to resist the tensile forces in those directions. Paragraph 19.4.12 is the only mention of spliced bars. It states: Splice lengths of shell reinforcement shall be governed by the provisions of Chapter 12, except that the minimum splice length of tension bars shall be 1.2 times the value required by Chapter 12 but not less than 18 in. The number of splices in principal tensile reinforcement shall be kept to a practical minimum. Since the containment is a shell, Chapter 19 of ACI 318 would allow lap splices. Also note that per Section 6.5 of calculation S10-0030, all lap splice lengths were increased by 20%.

ASME Code:

ASME Section III, Division 2, Section CC-3532 states: Where a non-pre-stressed reinforcement bar splice must be located in a region where tension is predicted in a direction perpendicular to the bar to be spliced, only a positive mechanical splice or a welded butt splice shall be used, unless calculations or tests of the selected splice detail are made to demonstrate that there is an adequate transfer of force. It should be noted that the Code is very unclear as to whether it is considering a member that is under biaxial tensile stress with tensile reinforcement placed in only one direction, or whether there is additional reinforcement that is perpendicular to the splice. In Bay 34, all mechanical and lap splices have reinforcement that is perpendicular to the splice, thus limiting the width of any crack that could occur parallel to a splice. Although Crystal River containment is not committed to ASME Code Section III, areas that have biaxial tension in Bay 34 were evaluated to demonstrate that adequate transfer of force exists in each area.

Results of Crack Width Testing in Containment Walls loaded in Biaxial Tension - Atomic Energy Control Board of Canada:

As previously noted, all mechanical and lap splices have reinforcement that is perpendicular to the splice, thus limiting the width of any crack that could occur parallel to a splice. This statement is supported by testing commissioned by the Atomic Energy Control Board of Canada involving prestressed wall segments simulating portions of containment walls loaded in biaxial tension that found no difference in crack widths for spliced bars relative to un-spliced bars. In addition, this testing found that transverse stresses had little effect on measured crack widths if controlled by reinforcement. The effect of transverse stresses was ignored in the crack width calculations with very little error. This testing involved concrete tensile strains as high as 0.002, which typically yielded the reinforcement and employed biaxial tensile stress ratios of 1:1 and 1:2 (MacGregor, J.G., Rizkalla, S.H., and Simmonds, S.H., “Cracking of Reinforced and Prestressed Concrete Wall Segments” (Reference 3).

New concrete tensile capacity normal to the lap splice:

Tensile stresses in a lap splice are transferred from one bar to the other through the surrounding concrete. Tensile stresses both parallel and normal to the splice could cause cracking in the surrounding concrete resulting in a reduction in the strength of the splice. The tensile capacity of the new concrete based on a specified compressive strength of 7000 psi is:

$3.0x \sqrt{f'c} = 251$ psi (FSAR, Section 5.2.3.3.1)

The acceptance criteria adopted in this response for lap splices that exist in a biaxial tension field are as follows:

(i) The maximum allowable bond stress (μu) is determined per the requirements of ACI 318-63, Section 1801. For ultimate strength design the bond stress μu for “top bars” has conservatively been assumed = $6.7 \times \sqrt{f'c}/D$ for “top bars” (for additional information concerning the correct interpretation of Section 1801 refer to Question 186).

(ii) The maximum allowable bond stress in a lap splice is determined per ACI 318-63, Section 805(b) and is = $0.75 \times$ μu

(iii) The maximum tensile stress in the concrete, normal to the splice, is limited to 251 psi assuming the splice is fully stressed to 0.9Fy. In situations where the tensile demand on the splice is less than 0.9Fy, the normal tensile stress limit may be increased upon further evaluation of the resultant stress field in the surrounding concrete. A predicted limit is the concrete splitting tensile strength.

Note: As described in Reference 3, much higher limits could be applied. This reference would imply that impact to bond stress does not occur until the concrete cracks and then the impact is trivial. Additionally, using the specified tensile capacity of 3.0 x √f'c is conservative for this application considering that the concept of biaxial tension as it applies to splice length is not part of the CR3 design basis.

Since the concern is the ability of the splice to transfer force from one bar to the other, ideally through uncracked concrete, using a more realistic value for the tensile strength of the concrete capacity would seem reasonable. The splitting tensile strength of concrete is between 6 and 7 x √f'c, and the modulus of rupture is 7.5 x √f'c (Refer to Reinforced Concrete Design, Wang and Salmon). This translates into a tensile strength ranging between 502 psi to 628 psi, based on 7000 psi concrete.

However, as previously noted, this response is based on the conservative assumption that the tensile strength of concrete =3.0 x √f'c = 251 psi. The exception to this is Area 2 above the equipment hatch where the tensile demand on the lap splice is considerably less than 0.9Fy, but the tensile demand normal to the splice exceeds 251 psi. A more detailed evaluation of the resultant stress field in this area has been provided.

Listed below, splices with biaxial tension are evaluated relative to this limit.

Areas of Biaxial Tension:
From a review of the membrane stresses (Attachment 1), extracted from the ANSYS analysis, in the hoop and vertical direction for Load Combination 1 which is the most critical load case for tension, the following areas in Bay 3 - 4 experience biaxial tension:

1. The area located above the equipment hatch between arc length 0.0' and 19.9'vertical in the vertical direction:

For both ease of construction and as a conservative measure, many of the existing #11 vertical reinforcement dowels extending out from the structure at Elevation 158' have been mechanically coupled with a continuous bar up to Elevation 176'-0". A total of 72 vertical #11s exist in this area (Refer to Attachment 3, Page 1 of 2 for as-built drawing of vertical splices in this area) of which 16 are lap spliced (primarily due to congestion) and the remaining 56 are mechanically spliced, i.e. 78% are mechanically spliced in the biaxial tension zone. The normal (hoop) tensile stress in this area ranges from 83 psi to 244 psi, is less than 251 psi, and will not affect the tension capacity of the lap splices.

2. The area located above the equipment hatch between arc length 0.0' and 19.9' in the horizontal direction (includes response to follow-up question):

In the hoop direction, from Elevation 159' to 165', new #10 bars were lapped spliced at the buttresses, where only uni-axial tension exists (Refer to Attachment 3, Page 2). Per Calculation S10-0030, "Containment Shell Design for the Construction Opening and Delaminated Areas" (Reference 1) the reinforcement provided in this area consists of #11s @ 12"c/c alternating with #10s @ 12"c/c (Total tensile area of reinforcement = 2.83 in2/ft) .

From Elevation 165' to 176', approximately 50% of the required hoop bars, i.e. 11- #10s at 12" were mechanically spliced to existing #10s at the buttresses (due to rebar congestion). These #10 bars are staggered lap spliced about 51" on either side of the bay 34 centerline in the biaxial tension zone (Attachment 2, Page 2 and Attachment 3, Page 2). The normal (vertical) tensile stress in this area ranges from 257 to 405 psi (Attachment 1, Page 3) which exceeds the acceptance limit of 251 psi. To better understand the resultant biaxial tensile stress field surrounding these lap spliced bars, the maximum predicted tensile stress in the lap splice is determined as follows:

Per Calculation S10-0030, Appendix C, the maximum bending moment in the hoop direction between Elevation 165' to 176' and from arc length 0.0' to 9.9' occurs in Cut #20 at Arc length 0.0' equals -50.41k-ft/ft for Load

Case #1 (0.95D + Fa + 1.5Pa + Ta) and produces compression on the outside face. Per Calculation S10-0030, Page 100, the moment capacity (Area of tensile reinforcement = 2.83 in2/ft) in the outer face in the hoop direction between Elevation 159' and 179' is 479 k-ft/ft which obviously exceeds the compressive moment of -50.41 k-ft/ft

The maximum bending moment in combination with axial tension (in the same area of interest) occurs at Cut #20 at arc length 0.0' is M = -50.41k-ft/ft and N = 92.58k/ft (tension) for Load Case #1. There is an outer layer of reinforcement as previously described (Area = 2.83 in2/ft) and an inner mat consisting of #11s @ 12"c/c (Area = 1.56 in2/ft). The total reinforcement available to resist axial (membrane) tension is 2.83 + 1.56 = 4.39 in2/ft. The resulting computed tensile stress in the #10s due to the axial tension can be determined as follows:

fs = (92.58 kips) / (4.39 in2/ft) = 21.08 ksi

The resulting tensile stress in the #10s due to bending stresses can be determined as follows:

Moment from Load Case 1 = -50.42 k-ft/ft (Compressive stress at the outer face)

Moment from accident thermal gradient = 111 k-ft/ft (See Section 6.1 in S10-0030)

Resulting moment in the section Mu = 111 – 50.42 = 60.58 k-ft/ft

ΦMn= ΦAs x fs x j d

As = 2.83 in2/ft, d = 39.045", ΦMn = 60.58 k-ft/ft

Assume j = 0.9 → 60.58 x 12 =0.9 x 2.83 x fs x 0.9 x 39.045

→ fs = 8122 psi = 8.1 ksi

Total tensile stress in the outer layer of rebar:

= 21.08 + 8.1 = 29.2 ksi < 0.9 fy = 54 ksi (54%)

ACI 318-63, Section 805(b) states that "the splice shall transfer the entire computed stress from bar to bar without exceeding three-fourths of the permissible bond stress of Section 1801; however, the length of the lap for deformed bars shall be not less than 24, 30 and 36 bar diameters for specified yield strengths of 40,000, 50,000 and 60,000 psi, respectively".

Conservatively assume these bars are "top bars":

The bond stress μ = 6.7 x √f'c/D, nor 560 psi (ACI 318-63, Section 1801)

The maximum bond stress to be transferred for a #10 @12"c/c is:

μmax = (0.75 x 6.7√7000)/(1.27) = 331 psi

Note that the maximum bond stress μmax = 331 psi is assumed as being fully developed at the acceptance limit for normal tensile stress of 3 x √f'c =251 psi.

The bond stress in the concrete (μ) based on a maximum tensile stress of fs = 29.2 ksi (as previously calculated) can be determined from the following equation:

L= As x fs/(μ∑o), where ∑o = sum of perimeters of effective bars (Reference 2)

Per Reference 1, Section 6.5, the lap splice length provided for #10 @ 12”c/c is 55”

 55 = (1.27 x 29200)/(μr x 4.0)

μ = 169 psi < 331 psi (51%)

In summation, the maximum tensile stress in the lap spliced #10 reinforcement is 54% of the maximum allowable tensile stress. The resulting bond stress between the #10 and surrounding concrete is 51% of the permissible bond stress as calculated per ACI 318-63, Section 805(b). Considering the low bond stress in the #10 bars (169 psi) and the fact that the transverse tensile stresses are not likely to crack the concrete since concrete strength in biaxial tension is the same as in uniaxial tension (Reference 4), these splices are acceptable. In addition, the Canadian research (Reference 3) found that the transverse tension (if controlled by reinforcement) had little effect on crack width, despite concrete strains that went to steel yield.

As stated above, acceptance criteria established for this response includes quantification of the impact of normal stresses to bond capacity. The following identity is used:

The maximum allowable bond stress μmax = 331 psi and is fully developed at the acceptance limit for normal tensile stress of 3 x √f'c =251 psi, and since these two stresses are orthogonal to each other, the relationship between the tensile stress (σ) and bond stress (μ) is assumed as follows:

("μ" /"331")^"2" "+" ("σ - 251" /"251")^"2" "=1" Equation 1

As shown in the graph below, this relationship is used to test combinations of bond stress and normal tensile stress. In the graph, the maximum tensile stress is set to the lower bounds of the concrete splitting tensile stress of 502 psi.

(see request folder for graph)

Based on Equation 1 the maximum allowable normal tensile stress = 467 psi acting in conjunction with the reduced bond stress of 169 psi which is greater than the normal tensile stress demand of 405 psi and is therefore acceptable.

3. The area between Elevations 216.9’ and 239.3’ and between arc length 0.0’ and 15’ in the vertical direction

The vertical splices on the outside face of the containment wall at Elevation 239.3 are lapped (noncontact splices) since existing #18 bar spacing does not match the spacing of the new #11vertical rebar cages (Attachment 2, Page 2). Additionally, #11 to #18 safety-related mechanical splices are not commercially available; therefore, mechanical splices cannot be employed in this area. Note that the normal (to splice) hoop tensile stress from Elevation 229’ to 239’ ranges from 176 psi in tension to 29 psi in compression, (Attachment 1, Page 2) is less than 251 psi, and will not affect the tension capacity of the lap splices.

4. The area (on the outside face of the containment wall) at the buttress/wall boundary between Elevation 218’-6” to 237’-9” in the horizontal direction:

The reinforcing bars in this area that are potentially affected are the horizontal bars, #9 & #8 coming out from the buttress between Elevation 220’ and 230’. The #9 bars are mechanically spliced per design (not affected) and the #8 bars are lap spliced with # 11 bars (Refer to Attachment 2, Page 2). The hoop tensile stress in this area is less than 76 psi (Attachment 1, Page 2) and the normal (to the splice) vertical tensile stress is less than 57 psi (Attachment 1, Page 3). These tensile stresses are significantly less than 251 psi in both directions and do not affect the tension capacity of the lap splices.

5. The area on the inside face of the containment wall in the 42” thick repair area in the vertical direction:

In this area, new vertical #11s must be spliced to existing #18s (Attachment 2, Page 4). Because the #18s do not line up with the #11s in either plane (the existing vertical #18s are further from the liner plate than the #11s), mechanical splices cannot be used and non-contact splices are used. The bottom of the non-contact splice is at Elevation 220' and the top of the splice is at approximate Elevation 230'. The normal (to the splice) hoop tensile stress in this area ranges from 30 psi to 176 psi (Attachment 1, Page 2), is less than 251 psi, and does not affect the tension capacity of the lap splices.

Conclusion:

Although ACI 318-63 is the code of record for Crystal River containment, the evaluations performed for each of the areas that experience bi-axial tension have demonstrated that the tensile capacity of the new concrete provides adequate transfer of force.

(see request folder for attachments and references)

Attachment 1:
Page 1: Areas of Biaxial Tension
Pages 2 & 3: Stresses at the 24" depth for the Critical Load Case

Attachment 2
Page 1: Outline of Repair Area
Pages 2 through 4: Rebar arrangements

Attachment 3
Pages 1 and 2: Mechanical splice location sketches

References:

1. Calculation S10-0030, Rev. 1, Containment Shell Design for the Construction Opening and Delaminated Area

2. Reinforced Concrete Design, Shaum Publishing Co., 1966.

3. J. G. MacGregor, S. H. Rizkalla, and S. H. Simmonds, "Cracking of Reinforced and Prestressed Concrete Wall Segments", Structural Engineering Report No. 82, University of Alberta.

4. Behavior of Concrete Under Biaxial Stresses by Helmut Kupfer, Hubert K Hilsdorf, and Hubert Rusch, ACI Journal, August 1969.

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/14/2010

Status: Closed

Date Closed: 12/17/2010

Request Number: 188

Individual Contacted: Steve Cahill

Date Contacted: 8/17/2010

Requestor/Inspector: Farhad Farzam

Category: Issue

Request: According to Page 42 of Calculation S10-0030, the interaction diagrams to check the section capacity against the demand due to the factored load combinations are developed using stress-strain distribution based on the ultimate strength design methodology. Section 5.2.3 of the CR-3 FSAR states the following: "The prestressed concrete reactor building has been designed to have a low strain elastic response to all design loads, thereby ensuring that the integrity of the vapor barrier is never breached. The design has been based upon various combinations of loads which have been increased by factors to approach the limit of an elastic response for the structure. These factors were developed in a similar manner to the ultimate strength design provisions of ACI 318-63."

Implementing the ultimate strength design methodology for the design of reinforcing steel in the repair area may not result in a low strain value in the reinforcing steel. Discuss whether this is a departure from the CR-3 design basis for the containment wall.

Follow-up request:

Explain in more detail the primary load case that was evaluated by the Working Stress Design (WSD) method that supports the statement that the design ensures an elastic, low strain response to the normal operating loads.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

As required by the CR3 FSAR, Section 5.2.3, the reinforcement requirements for the containment shell were verified for normal operating loads per the Working Stress Design (WSD) method and for accident loads per the Ultimate Strength Design (USD) method. The WSD check ensures an elastic, low strain response to the normal operating loads. Consistent with FSAR, Section 5.2.3.2.1, overload factors were applied to the design loads which were then combined per the FSAR into design basis accident load combinations and applied to the ANSYS finite element model. (FEM) The FEM of the CR3 containment shell is solved by the methods of elastic analysis, i.e. stress is proportional to strain, which ensures that the load deformation behavior of the overall structure remains one of elastic, low strain response. The FEM did not include the cracked state of the concrete, i.e. an elastic, linear solution is utilized by the program. The strength of individual sections of the structure is then evaluated using the USD method as described in FSAR Section 5.2.3 which includes a strength reduction factor Φ per FSAR Section 5.2.3.3.1. The USD check for accident loads does not imply that any material has yielded under service load conditions since the service loads have been increased by load factors when checking the structure for ultimate strength capacity.

Review of available original design basis calculations clearly document use of the USD method; for instance, USD method was employed for the reinforcement around penetrations, the containment basemat and basemat to wall interface design. Therefore, the design of the reinforcing steel for the repair area is consistent with the CR3 original design basis load combinations as required by CR3 FSAR, Section 5.2.3.

Response to follow-up request (11/8/10):

Calculation S10-0030, Reinforcement Design for Delaminated Containment Wall (Reference 1), Section 2.0 lists the WSD load combinations that were evaluated. These loads were:

Service Load Combinations:
6- 0.95D + Fa + 1.0Ex + 1.0Ev + To
7- 0.95D + Fa + 1.0W + To
8- 0.95D + Fa + 1.0P + Ta
9- 0.95D + Fa + 1.0Wm + 1.0Wp + To
10- 0.95D + Fo + 1.15P
11- 1.05D + Fo - 1.0Ex - 1.0Ev + To
12- 1.05D + Fo + 1.0Ex - 1.0Ev + To

Where:
Fa = Prestress load at end of life
Fo = Prestress load at return to service
P = Design accident pressure load
Ta = Thermal loads based upon accident temperature
W = wind load based on design wind
Wm= wind load based on tornado
Wp = Internal pressure due to tornado
D = Dead load
Ex = Lateral seismic load based on 0.05g ground motion (Operating Basis Earthquake)
Ev = Vertical seismic load based on 0.05g ground motion (Operating Basis Earthquake)

Reference 1, Section 6.3.2 states that Load Combination #8 was the limiting load case for axial tension and bending moment. This load combination is not specifically listed in FSAR Table 5.3, however, FSAR Section 5.2.3 states; The prestressed concrete reactor building has been designed to have a low strain elastic response to all design loads… and FSAR Section 5.2.3.3.1 states; the concrete shell has been prestressed sufficiently to eliminate tensile stresses due to membrane forces from design loads. Since accident pressure is a design load (reference FSAR Section 5.2.1.2 “Design Loads”) which must be combined with prestress to ascertain its effect on the concrete shell, Load Combination 8 was conservatively created, to include accident temperature and was evaluated by the WSD method. Additionally, the following FSAR section supports this approach:

FSAR Section 5.2.5.2.1.1, 5th paragraph states; In addition to the load combination described in Section 5.2.3.2 where design is based upon an "Ultimate Strength Design" approach, the reactor building was also designed to accommodate construction and the controlling operating load combinations in accordance with ACI 318-63 "Working Strength Design" and "Prestressed Concrete”.

Reference 1, Section 6.3.2.3 and 6.4.2.3 evaluate the concrete shell in Bay 34 in the vertical and hoop directions for axial tension and bending moment resulting from Load Combination 8 by WSD methods. When using WSD techniques, members are proportioned/evaluated so that they may sustain the anticipated real induced loads (design loads) without the stresses in the concrete or reinforcing exceeding the proportional limits of the individual material, thus ensuring an elastic, low strain response to the design loads (normal operating loads).

Misc Notes:

Response By:	Don Dyksterhouse		
Reviewed By:		**Date Response Provided:**	9/7/2010
Status:	Closed	**Date Closed:**	11/10/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:39 AM

Request Number: 189

Individual Contacted: Steve Cahill

Date Contacted: 8/17/2010

Requestor/Inspector: Farhad Farzam

Category: Issue

Request: Discuss whether the maximum compressive stresses determined in Calculation S10-0012, Revision 1 and Calculation S10-0015, Revision 0, are in compliance with the CR-3 design basis concrete compressive allowable stresses taking into account the net area of the wall.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

The effect of the tendon areas is evaluated in the design; however, as shown in Section 6.3.1.4 of calculation S10-0030 (Reinforcement Design for Delaminated Containment Wall), any potential effect is negligible. The area occupied by a vertical tendon per one foot of the wall in the circumferential direction is 1.4% of the total area and the area occupied by a hoop tendon per one foot of the wall in the vertical direction is 2.7% of the total area. These small reductions in cross-sectional area have been ignored in calculation S10-0030 which is consistent with methods utilized in the original design (FSAR Section 5.2.5.2.1.1(b).\

A compression check for the compressive stresses is provided in Calculation S10-0030. Section 6.3.1.4 includes a check for the compressive stresses in the vertical direction and due to factored load combinations. Section 6.3.2.2 includes a check for the compressive stresses in the vertical direction and due to service load combinations. Section 6.4.1.2 includes a check for the compressive stresses in the hoop direction and due to factored load combinations. Section 6.4.2.2 includes a check for the compressive stresses in the hoop direction and due to service load combinations.

Reference:
S10-0030 Reinforcement Design for Delaminated Containment Wall

Misc Notes:

Response By: Don Dyksterhouse

Reviewed By:

Date Response Provided: 9/7/2010

Status: Closed

Date Closed: 10/29/2010

Request Number: 190

Individual Contacted: Paul Fagan

Date Contacted: 8/10/2010

Requestor/Inspector: Frahad Farzad

Category: Question

Request: Calculation S10-0030 rev. 0

If your investigation determines that request #183 and request #184 are technical errors that should have been identified during the verification process according to your quality assurance program and independent reviews of the calculation, provide further information on the corrective actions that you plan to take to insure technical adequacy of all relevant design basis calculations that are performed for CR3 containment structure repair.

References:

Response Assigned to: Don Dyksterhouse

Date Due to Inspector:

Response:

Responses to questions #183 and #184 regarding potential technical errors in S10-0030, Reinforcement Design for Delaminated Containment Wall, (AREVA Calculation 32-9141877) are as follows:

Q183: A technical error was made in selecting the working stress limit of 40,000 psi for the existing stirrups. Using guidance in ACI 318-63 Section 1003, a stress limit of 20,000 psi for the existing stirrups should have been used.

Q184: A technical error was not made in determining the value for Vci. The extremely large numbers in the calculation were due to a small number being used in the denominator. This high number was not used because this value was not the smaller of two numbers was used as specified in ACI 318-63 Section 2610. Although the calculation will be clarified to address the high values, the correct value was used in the calculation.

To address the error in the working stress limit for the existing stirrups, two corrective action documents were initiated:

1. AREVA WebCap 2010-5579 was initiated and an independent extent of condition was performed to review other ACI 318 Code applications in AREVA calculations that have been made in support of the Crystal River Containment Repair Project.

2. Progress Energy NCR 416423 was initiated to address correcting the shear limit of 40,000 psi that was used for the existing stirrups. Two corrective actions were initiated:
   i. An action item (Assignment 5) was created to correct the error in Calculation S10-0030. Calculation S10-0030 is being revised (revision 2) to address this corrective action.
   ii. An action item (Assignment 6) was created to validate that the extent of condition performed in Webcap 2010-5579 adequately addressed the error in using the incorrect working stress limit for the existing stirrups. This corrective action remains open.

Summary:
An extent of condition has been performed by AREVA in WebCap 2010-5579 and has concluded that no additional incorrect applications of ACI 318-63 in AREVA generated calculations. The effectiveness of this extent of condition is being evaluated by Progress Energy in NCR 416423 assignment 06.

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:39 AM

Misc Notes:

Response By:	Don Dykstergouse		
Reviewed By:		**Date Response Provided:**	11/10/2010
Status:	Closed	**Date Closed:**	3/1/2011

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:39 AM

Request Number: 191

Individual Contacted: Steve Cahill **Date Contacted:** 10/18/2010

Requestor/Inspector: Meena Khanna **Category:** Information Request

Request: The root cause report of the CR3 containment wall delamination event identified, among others, the following as contributing failure modes to the delamination:

(1) The presence of small pre-existing cracks (of the order of 5 mm) around the hoop tendon sleeves, approximately in the vertical plane of the hoop tendons, from the original tensioning of tendons which propagated into the larger delamination due to redistribution of stresses from the scope and sequence of detensioning used for creating the SGR construction opening. These cracks resulted from significant stress concentrations around the periphery of the hoop tendon sleeves from prestressing forces in the hoop and vertical tendons. These small cracks potentially exist around all existing hoop tendon sleeves outside of the delamination repair area.

(2) Relatively higher prestress level in the CR3 containment design compared to other similar post-tensioned containments in the industry. Note that the 155 hoop tendons that were detensioned for the delamination repair and all the 144 vertical tendons are being retensioned and the minimum design tendon forces for hoop and vertical tendons are being significantly increased, as part of the design for the CR3 containment wall delamination repair.

In light of the above facts, please provide information on what measures are being taken prior to, during, and following retensioning of tendons to (a) prevent and (b) physically verify that a propagation of the pre-existing cracks into a delamination or a new delamination has not occurred.

References:

Response Assigned to: Don Dyksterhouse **Date Due to Inspector:**

Response:

(a) Prevention: Prior to retensioning, PII has individually modeled each tendon in both the NASTRAN (linear displacement & stress model) and ABAQUS (visco-elastic fracture energy model) models. These models have been calibrated to the CR3 delamination history events and have correlated well with one another. The Abaqus model reflects the presence of pre-existing cracks and has the capability at the "microscope" level to monitor and identify crack propagation. The final retensioning sequence will be evaluated and optimized by PII to verify that a propagation of the pre-existing cracks into a delamination or a new delamination has not occurred based on fracture energies. PII's efforts will also review stresses at these locations on the top and bottom of the hoop tendons. Any significant increases against established criteria in the calibrated models will be dispositioned to allow adequate margin prior to finalizing the retensioning sequence. Tendon forces include the overstress condition (80% GUTS) which envelops lock-off values.

Cracks that formed at the top and bottom of hoop tendons are due to high localized stresses. Any of these localized stresses that exceed the tensile capacity of the concrete are self-relieving. After the cracks are formed, the stress decreases and crack growth is arrested. For hoop tendons that do not have pre-existing cracks, potential cracking similar to what was observed and documented in the root cause report may occur during retensioning. These "new" cracks would also be self-relieving and crack growth would also be arrested. For hoop tendons that have pre-existing cracks, the stress has already been relieved. As noted above, the Abaqus model has the ability to monitor for propagation of these cracks. Significant engineering effort is involved in development of a retensioning plan to ensure no delamination occurs due to this stress relief mechanism or other effects (supplemental sentence 11/29/10).

(b)Verification: No monitoring will occur prior to retensioning. Significant monitoring of containment will occur during retensioning, testing and recovery activities. The details below are actions to address delamination cracking.

Strain gauges: During retensioning, strain gauges have been embedded in Bay 34 that will provide data on the new concrete at various locations. Additionally, a strain gage is located across the plane of maximum radial tension in Core Hole No. 72 in Bay 23 (See response to S.I.T. Question 146 for details on strain gauges)

Acoustics: Acoustic instruments will be monitored under the guidance of MISTRAS Group during retensioning. Acoustic Emissions sensors will be monitored during the retensioning process of the containment building, similar to the monitoring that was performed during the detensioning process. Monitoring will be performed under the guidance of trained, experienced individuals from the MISTRAS Group. Any indications or precursors of cracking during retensioning will trigger an immediate stop of retensioning work and the tendon ram will be placed in a safe condition. A review of this additional cracking must be evaluated prior to proceeding with additional work. Bay 23 and Bay 45 have high density sensor arrays. All bays, other than Bay 34, will have multiple single sensor locations. Locations of acoustic sensors are identified in EC 75221 Attachment Z28 but are subject to change as a result of development of retensioning sequence. (Reference 2)

Impulse response (IR): After retensioning, IR scans will be taken in area of high stress. Panels in each bay, other than Bay 34, will be scanned. Although the IR scan will typically not identify small cracks as currently observed on the top and bottom of the hoop tendons, any significant growth or propagation of these cracks would typically be identified. For example, if the cracks at the bottom and top of the closely spaced tendons were to grow and connect, it is likely that the IR scan would identify this significant increase in crack growth. Identification of any cracking in the vicinity of the hoop tendons would be identified and evaluated prior to startup. Baseline IR scans are also being performed for areas required to be inspected in future surveillances in response to the Delamination Root Cause (Ref. NCR 358724-32). These scans are performed after pressure tests.

References:
Reference 1: EC 75221 Section E Testing Requirements
Reference 2: EC 75221 Attachment Z28 Acoustic Monitoring Sketches
Reference 3: EC 75221 Section B.6.20 Testing Requirements

Misc Notes:

Response By:	Paul Fagan		
Reviewed By:		**Date Response Provided:**	11/4/2010
Status:	Closed	**Date Closed:**	12/6/2010

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:39 AM

Request Number: 192

Individual Contacted: Steve Cahill **Date Contacted:** 2/23/2011

Requestor/Inspector: Louis Lake **Category:** Question

Request: FSAR Section 5.2.2.3 “Post-Tensioning System” states that each tendon used at CR3 consists of 163 7-mm diameter low relaxation wires and developed a minimum ultimate tendon force of 2333.5 kips. Further the FSAR section states that the low relaxation wire conformed to the applicable portions of ASTM A 421-65, Type BA with a minimum ultimate tensile stress of 240 ksi. All design calculations (e.g. Calculations S10-0012, S10-0028, S10-0058, etc.) performed for the CR3 Containment Delamination Repair Project have been based on a guaranteed ultimate tensile strength (GUTS) of 240 ksi for the tendon wire.

Standard Specification ASTM A421-76 and later versions explicitly specify the minimum ultimate tensile strength of 235 ksi for 7.01mm (0.276 in) diameter Type BA wires. Versions earlier than that note against the 7.01 mm diameter wire that this size is not commonly furnished in Type BA wire, but specify 235 ksi for the corresponding Type WA wire. The calculations on pages 1.01.7/11 through 1.01.7/18 in CR3 Original Design Calculation 1.01.7, “Reactor Building Design – Prestressing” for vertical and hoop tendons document and use the value of ultimate strength of prestressing steel, fs’, as 235 ksi for the Type II tendon (163 – 0.276” diameter low relaxation (4%) foreign stabilized wire), which is consistent with the ASTM Specification A421. The calculations on page 1.01.7/49 for dome prestressing, however, uses the value of fs’ as 240 ksi for the low relaxation foreign stabilized wire, which seems arbitrary and not consistent with that used for vertical and hoop tendons for the low relaxation stabilized wire.

(a) Explain the discrepancy in the value of GUTS for tendon wire used in the CR3 design calculations for the tendon wire size and material conforming to ASTM A421 Type BA for the different tendon types (vertical, hoop and dome). Provide a copy of ASTM A421-65, if available.

(b) Was the value of GUTS of 240 ksi an error? If so, how would you correct it and what is its impact on the applicable CR3 containment design calculations, Engineering Change packages, and tendon surveillance calculations? The NRC staff notes that a change in the value of GUTS affects all the allowable stresses/forces in the tendons.

(c) If not an error, provide documentation that guaranteed the minimum ultimate tensile strength (GUTS) of the tendon wire as 240 ksi for: (i) the original tendons, and (ii) the tendons being replaced by new ones in containment delamination repair project.

(d) Is the dome tendon wire different (7mm vs ¼” and stress-relieved vs stabilized) from the hoop and vertical tendon wires. The reason for the question is that Section 4.4.4 of the dome delamination report uses the yield strength of tendon wire as 0.8 time GUTS (vs 0.9 GUTS for low relaxation wire).

References:

Response Assigned to: Ron Knott **Date Due to Inspector:**

Response:

a) All tendons use 240 ksi, low relaxation, 7 mm wire. The final design uses this wire. The referenced calculation pages are preliminary design that was used to select prestress system components for CR3 and another proposed facility. Note: The wire type discrepancy was also identified during the root cause investigation for the SGR delamination event. Reference NCR 378555. As requested, ASTM A421-65 was provided.

b) 240 ksi is correct.

c) Surveillance wire testing results are all above 240 ksi and meet 4% elongation requirements. A sample from the 30th year surveillance is attached. New tendons also use 240 ksi wire. A sample of wire testing is attached.

d) All tendons are the same. The Dome Delamination Report is in error. See attached review comment and response from Worley Parsons regarding 80% or 90% fpu. Note: Tested values for yield stress are higher than 90% of specified ultimate strength. See attached 30th Year Surveillance Report for a sample. All yield values are greater than 0.90 x 240 ksi = 216 ksi.

Reference:
NCR 378555 – "DBD 1/1, Containment, Has an Incorrect Value for Tendon Wire" 1/29/2010

Attachments (see response folder for attachments)
1. 30th Surveillance Report – Wire Testing
2. EC 63016 – SGR Wire Test
3. Calculation S10-0050 – Owner Acceptance Review

Misc Notes:

Response By: Ron Knott

Reviewed By:

Date Response Provided: 2/28/2011

Status: Closed

Date Closed: 4/5/2011

2009-10 NRC Special Inspection – CR3 Reactor Building Concrete Separation

20-Apr-11 7:56:39 AM

Request Number: 193

Individual Contacted: Steve Cahill

Date Contacted: 2/23/2011

Requestor/Inspector: Louis Lake

Category: Question

Request: Calculation S10-0028, Rev 1, "Containment Repair Project – Reactor Building Tendons / Forecast End of Life Force," considers a best-estimate of short term and time-dependent losses in estimating the 60-year end-of-life tendon forces and the increased minimum design tendon forces for tendons affected by the repair. It is recognized that each of the factors affecting the time-dependent characteristics of tendon forces are subject to variations. However, no potential variations in were considered in the calculation. These variations were evident at CR3 from the results of specifically the 6th, 7th and 8th tendon surveillances, where a significant number of tendons did not meet the IWL acceptance criteria primarily because the predicted curves significantly under-estimated time-dependent losses and especially the losses due to concrete creep. The original CR3 containment concrete has a significantly high creep with the ultimate creep coefficient of the order of 2.4 (see CR3 containment delamination root cause analysis report).

(a) Calculation S10-0028 estimates creep losses following repair based on a creep recovery of 25% of the elastic strain of original concrete , which is considered to occur over a 90-day period between detensioning and retensioning (see Section 4.3 and Attachment 4. The calculation is based on detensioning completed in March 2010 retensioning occurring in July 2010. However, actual retensioning is not expected to be complete until around March 2011, which is approximately a one-year period since detensioning during which the creep recovery is expected to be higher than that in 90-days. The methodology used in Attachment 4 is also expected to provide a higher estimate when a 1-year period is considered. What is the quantitative creep recovery during a 1-year period and its impact on the estimated creep losses? How do you justify use of a 90-day creep recovery in the calculation when the actual recovery took place over a 1-year period?

(b) Calculation S10-0028 estimates the creep and shrinkage losses for all hoop and vertical tendons using properties of the original concrete on the basis that the new concrete comprises 5% of the total volume. The NRC staff notes that for the hoop and vertical tendons that traverse the concrete repair area, the new concrete volume is significantly higher than 5%. Provide, in quantitative form, the impact of consideration of the new concrete properties and potential differential creep between new and old concrete on the tendon prestress losses due to creep and shrinkage for the tendons that traverse the new concrete.

(c) How does the increased minimum design tendon forces for the hoop tendons affected by the containment delamination repair project and all the vertical tendons comply with the allowable stress requirement for effective prestress in Section 2606(b) of ACI 318-63 (code-of-record)?

References:

Response Assigned to: Ron Knott

Date Due to Inspector:

Response:

a) The correct reversal value has been calculated to be 29% based on the actual durations and dates for re-tensioning. S10-0028 conservatively assumes the reversal to all occur after retensioning. Since hoops are tensioned twice (50% followed by 100% lock-off), a significant amount of the reversal occurs before final lock-off. These two small effects are offsetting and have been planned to be addressed in the update of S10-0028 for future surveillances when final lift-off forces are included.

b) The 5% effect refers to the entire structure and was not intended to be a localized application. S10-0028 determines the mean tendon force by group for the entire structure. Creep is not expected to have a significant effect because the mix design for the new concrete was developed for this specific purpose. The response to SIT Question 145 (Supplement 1) previously addressed this as less than a 5% effect. At that time estimated creep recovery was 28%. Use of 29% does not affect the result.

c) The new minimum required prestress is based on the exact same jacking force and tendon lock-off forces as original design. There is no change and therefore no impact to the requirements of ACI 318-63 Section 2606(b). The '71 revision of the code recognizes that the lock-off stress must also have an adequate safety factor under service conditions and cannot be considered as a temporary stress. The '71 code corrected an inconsistency in the '63 code by clarifying that a long term allowable tendon stress of 0.7 f`s is acceptable and provides a sufficient safety margin. The commentary to the '71 code states:

The Code no longer distinguishes between temporary and effective steel stresses, as did the 1963 Code. The reasoning is that the tendon stress immediately after transfer can prevail for a considerable time, even after the structure has been put into service. This stress, therefore, must have an adequate safety factor under service conditions and cannot be considered as a temporary stress. Any subsequent stress drop in the steel due to losses can only improve conditions and, hence, no allowable limit on stress drop has been provided in the Code.

Note also that this section of the code applies only to tendon stress. The original and replacement tendons were subjected to a test program consistent with intended design provisions. Other components in the CR3 prestress system such as concrete and anchorage steel implement 0.7 f`s as a service load and the overstress force (0.8 f`s) as a temporary load.

Misc Notes:

Response By: Ron Knott

Reviewed By:

Date Response Provided: 2/28/2011

Status: Closed

Date Closed: 4/5/2011