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February 27, 2012
L-12-065

Ms. Cynthia D. Pederson, Acting Administrator
United States Nuclear Regulatory Commission
Region III
2443 Warrenville Road, Suite 210
Lisle, IL 60532-4352

SUBJECT:
Davis-Besse Nuclear Power Station, Unit 1
Docket Number 50-346, License Number NPF-3
Submittal of Shield Building Root Cause Evaluation

On December 2, 2011, the Nuclear Regulatory Commission (NRC) issued Confirmatory Action Letter 3-11-001 (ADAMS Accession No. ML11336A355) regarding the identification of cracks in the reinforced concrete shield building at the Davis-Besse Nuclear Power Station (DBNPS). One requirement of the Confirmatory Action Letter is for the FirstEnergy Nuclear Operating Company (FENOC) to provide the results of the root cause evaluation and corrective actions to the NRC, including any long-term monitoring requirements.

The root cause evaluation of the DBNPS Shield Building cracks has been completed, and a copy of the Root Cause Analysis Report is enclosed. This report includes the corrective actions being taken along with long-term monitoring requirements.

There are no regulatory commitments contained in this letter. If there are any questions or if additional information is required, please contact Mr. Patrick J. McCloskey, Manager, Site Regulatory Compliance, at (419) 321-7274.

Sincerely,

Barry S. Allen

Barry S. Allen

GMW

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Enclosure: Root Cause Analysis Report: "Concrete Crack within Shield Building Temporary Access Opening"

cc: NRC Document Control Desk
DB-1 NRC/NRR Project Manager
DB-1 Senior Resident Inspector
Utility Radiological Safety Board

FirstEnergy Nuclear Operating Company
Davis-Besse Nuclear Power Station

Root Cause Analysis Report

Concrete Crack within Shield Building Temporary Access Opening

CR Number 2011-03346, Dated 10-10-2011

Post-CARB, 02-27-2012

Signed: ______________ **Date:** 2/27/2012
Certified Root Cause Evaluator – Kevin Browning

Signed: ______________ **Date:** February 27, 2012
Team Leader – Alfred J. Cayia

Approved: ______________ **Date:** 2/27/12
Owning Manager – Jon Hook

Approved: ______________ **Date:** 2/27/2012
Sponsor – Ken Byrd

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List of Acronyms

ACI	American Concrete Institute
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
CANDU	Canadian deuterium uranium nuclear plants
CARB	Corrective Action Review Board
CF	Causal Factor
CFD	Computational Fluid Dynamics
CR	Condition Report
CTE	Coefficient of thermal expansion
CTL	Construction Technology Laboratory
DCM	Design Criteria Manual
DBNPS	Davis-Besse Nuclear Power Station
ECP	Engineering Change Package
EDG	Emergency Diesel Generator
EPRI	Electric Power Research Institute
F	Fahrenheit
FEA	Finite Element Analysis
FENOC	FirstEnergy Nuclear Operating Company
FM	Failure Mode
FMA	Failure Modes Analysis
IAEA	International Atomic Energy Agency
INPO	Institute of Nuclear Power Operations
IR	Impulse Response
NRC	Nuclear Regulatory Commission
PCAQ	Potential Condition Adverse to Quality report
PII	Performance Improvement International
Rebar	Reinforcing steel
SEM	Scanning Electron Microscope
USAR	Updated Safety Analysis Report
USBR	United States Bureau of Reclamation

1 Abstract

On October 10, 2011 a concrete crack was observed at the architectural flute shoulder region of a temporary access opening in the wall of the shield building at the Davis-Besse Nuclear Power Station. The temporary access opening was being cut by supplemental personnel under the direction of FENOC using a hydrodemolition process to allow replacement of the reactor pressure vessel head. A concrete crack in the architectural flute shoulder region of a temporary access opening in the shield building wall was unexpected and needs to be understood to evaluate the impact on the shield building structural integrity currently, previously, or within the future viable service life of the plant. Previous visual inspections of the shield building exterior surface performed over many years did not identify any unusual surface defects or symptoms of distress that would have indicated the presence of the subsurface concrete laminar cracking.

A team of subject matter experts knowledgeable in concrete structure design, construction, examination, and modeling was assembled to perform an in-depth investigation utilizing visual inspection, examination of shield building core bore samples, and extensive computer modeling and analysis to determine the most likely failure scenario and the cause(s) of the observed condition. This root cause investigation is intended to determine "how," "when," and "why" the concrete laminar cracking occurred in the shield building wall.

Acoustic sounding of the shield building exterior wall was performed using the Impulse Response testing method to locate areas with concrete laminar cracking. Confirmation of the Impulse Response testing results was achieved by visual inspection of 70 core bores, including crack characterization. The initial condition assessment determined that the shield building concrete wall contained tight width laminar cracking near the outer face of structural reinforcing steel. The majority of the shield building laminar cracking occurred in the concrete at the outer face of structural reinforcing steel located behind the architectural flute shoulder region. Some laminar cracking occurred beyond the architectural flute shoulder region as evident across the top 20 feet of the shield building and in localized areas adjacent to one side of each main steam line penetration blockout. The southwestern exposure of the shield building wall was observed with the most extensive concrete cracking. The initial condition assessment determined that the shield building was functional, but non-conforming with the concrete laminar cracking.

Examination of 36 shield building concrete cores was performed to define possible failure modes for the laminar cracking, or quantify material properties of the concrete to support computer modeling and analysis. Two laboratories performed petrographic examination of four concrete cores in order to determine the concrete condition, possible reasons for the damage, and prediction of whether deterioration may continue. Four other laboratories examined the shield building concrete cores for material properties and possible failure modes.

The external laboratory examination of the shield building concrete cores determined that the concrete was in good condition, consistent with the mix design, and revealed no unacceptable or degraded material properties. There was no evidence of typical concrete time-dependent aging failure modes. The examination found the outer surface of the cores was not water-repellant, and the air voids were lined with secondary deposits of ettringite (crystal formation from sulfate reaction with calcium aluminates) and calcium hydroxide suggesting long-term exposure to moisture migrating through the concrete. The integrity of the concrete narrowed the failure mechanism to those related to design or environmental issues versus construction.

Computer modeling of shield building loads under environmental conditions with extreme combinations of temperature and wind showed that those combinations were insufficient to result in laminar cracking of the concrete. Therefore, the forces involved with the laminar cracking were beyond those anticipated with generally accepted design practices. The acute freezing and expansion of moisture in the shield building concrete was the only scenario capable of generating radial stresses large enough to initiate laminar cracking. The blizzard of 1978 was determined to be the only event during the life of the shield building that integrated the moisture content, wind speed, temperature, and duration necessary for development of radial stresses large enough to enable the concrete laminar crack initiation.

The reason for the shield building laminar cracking was the design configuration of the architectural flute shoulders coupled with a rare combination of severe environmental factors associated with the blizzard of 1978.

The design configuration did not include an exterior protective sealant on the shield building which allowed moisture to migrate into the concrete, freeze, and expand.

The design configuration inherent stress concentration at the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder enabled the radial stress from the freezing moisture to exceed the tensile strength of the concrete and initiate a crack. Other horizontal (hoop) and vertical stresses that adjoined the outer face of structural reinforcing steel underneath the architectural flute shoulder region enabled the laminar crack created by freezing moisture to propagate along the outer face of structural reinforcing steel.

The design configuration did not include radial reinforcing steel ties or stirrups at intermediate spacing between each end of the architectural flute shoulder reinforcing steel connection with the structural reinforcing steel which enabled the laminar crack created by freezing moisture to propagate.

The design configuration density of structural reinforcing steel within the top 20 feet of the shield building, and at the main steam line penetration blockouts (spacing was less than or equal to six inches) enabled laminar cracks created by freezing moisture to propagate.

The laminar cracking at the main steam line penetration blockouts is a result of the adjacent laminar cracking in the architectural flute shoulders that continued to propagate into the adjacent areas of higher density reinforcing steel. The laminar crack continued into the area at the main steam line penetration blockouts until obtaining equilibrium and stopping. Note that the bottom of the shoulders in this area is three feet above the auxiliary building roof and in the area of higher density reinforcing steel.

The southwestern exposure of the shield building wall was observed with the most extensive concrete cracking. The prevailing wind direction occurs from the southwest for most of the year, particularly during the winter and spring seasons, and was the path of the blizzard of 1978.

An analysis was performed of a shield building wall section with a simulated crack. This analysis evaluated the potential for extreme summer or winter temperature events to generate radial stresses sufficient to propagate the existing crack. This analysis concluded that the extreme temperature events do not cause stress levels that exceed the tensile strength of the concrete. Therefore, the existing laminar cracks are not expected to propagate.

The conclusion of this investigation is that the cause of the concrete laminar cracking was the design specification for construction of the shield building that did not specify application of an exterior sealant from moisture. The action to prevent recurrence of the shield building concrete laminar cracking is to apply an exterior protective sealant as a barrier against moisture migrating into the concrete. Therefore, with an effective exterior protective sealant the shield building concrete laminar cracking will not repeat under the required combinations of extreme environmental conditions such as the shield building experienced during the severe blizzard of 1978. The other nuclear safety-related structures on-site have a protective sealant as a barrier against moisture migration into the concrete.

2 Introduction

2.1 Problem Statement

On October 10, 2011 a concrete crack was observed at the architectural flute region of a temporary access opening in the shield building wall at the Davis-Besse Nuclear Power Station (DBNPS). The temporary access opening was being cut by supplemental personnel under the direction of FENOC using a hydrodemolition process to allow replacement of the reactor pressure vessel head. Various visual inspections of the shield building exterior surface performed over many years did not identify any unusual surface defects or symptoms of distress that would signify the presence of the subsurface concrete laminar cracking.

2.2 Consequences

A concrete crack in the architectural flute region of a temporary access opening in the shield building wall was unexpected and an investigation was started to understand the condition of the shield building and determine how and why the observed condition occurred. Additionally, the initial investigation evaluated the observed condition and determined that the shield building was capable of performing its intended function prior to restart of the unit from the mid-cycle outage. The investigation described in this report determined the most likely failure scenario, identified cause(s), and determined the corrective action(s) to ensure the shield building structural integrity for the remaining service life of the plant including extended operation in the license renewal period.

2.3 Self-Identification Information

Previous inspections of the shield building exterior surface did not identify symptoms that would signify the presence of the concrete sub-surface laminar cracking.

2.4 Root Cause Team Members

A team of industry experts was contracted by FENOC to support the investigation and determine the cause(s) and recommended actions. The team was selected based upon capability and prior experience in root cause investigation with particular emphasis on previous industry experience in nuclear containment laminar cracking.

Alfred J. Cayia, Director – Fleet Performance Improvement and Root Cause Team Lead

Kevin Browning, DBNPS Root Cause Evaluator

Richard Bair, DBNPS Civil / Structural Engineer

Thomas Henry, DBNPS Civil / Structural Engineer

Performance Improvement International (PII) was the prime contractor with prior industry experience in both root cause investigation and modeling and analysis capability of nuclear containment structures.

Performance Improvement International

Dr. Chong Chiu	Dr. Avi Mor	Dr. Mostafa Mostafa
Dr. Ray Waldo	Dr. Yunping Xi	Joe Amon
David Dearth	Tyson Gustus	Henric Larsson
Doug Marx	Luke Snell	Doug Starck
Hany Helmy		

During the early stages of the root cause investigation, two additional firms with experience in civil structural evaluation of nuclear concrete structures and with root cause investigative expertise participated in the development of potential failure modes for the observed condition. This approach ensured that a comprehensive list of failure modes was developed to be further evaluated during the course of the investigation.

MPR Associates

Edward Bird	James Bubb	Richard Stark
Dave Werder		

Vatic Associates, Inc.

Steve Eisenhart

3 Data Analysis

3.1 Methodology

A team of subject matter experts knowledgeable in concrete structure design, construction, examination, and modeling were assembled to review evidence and documentation associated with the indication of a concrete crack in the architectural flute region of a temporary access opening in the shield building wall. The observed cracking of the shield building wall was unexpected and required rigorous investigation to be understood to ensure there is no impact with its structural integrity for the future viable service life of the plant. This root cause investigation is intended to determine "how," "when," and "why" the concrete cracking occurred in the shield build wall.

This root cause investigation was conducted using the guidance described in procedure NOP-LP-2001 (Corrective Action Program), business practice NOBP-LP-2011 (FENOC Cause Analysis), and reference material NORM-LP-2003 (Analytical Methods Guidebook). A pre-job brief was conducted with the owning Manager, and a task assignment was approved by the sponsoring Director. The root cause investigation was performed using the TapRoot® methodology along with Problem Solving and Decision Making (procedure NOP-ER-3001), Equipment Apparent Cause Evaluation (form NOP-ER-1001-01), Event & Causal Factors Charting, Barrier Analysis, Change Analysis, and Fault Tree Analysis.

The primary contractor supporting the root cause investigation, Performance Improvement International, is a recognized expert in root cause analysis. The Performance Improvement International methodology is one of the techniques endorsed by FENOC to perform root cause investigations.

The following additional Condition Reports discovered during the initial problem review were rolled into this root cause investigation:

2011-04214, Core Bore Found Additional Crack in Architectural Flute Area (10/24/2011)

2011-04402, Fractured Concrete Found at 17M Shield Building at Main Steam Line Penetrations (10/26/2011)

2011-04648, Shield Building Impulse Response Indications Above Elevation 780 (10/31/2011)

2011-05475, Concrete Cracking at the Top of the Shield Building Wall (11/16/2011)

3.2 Sequence of Events

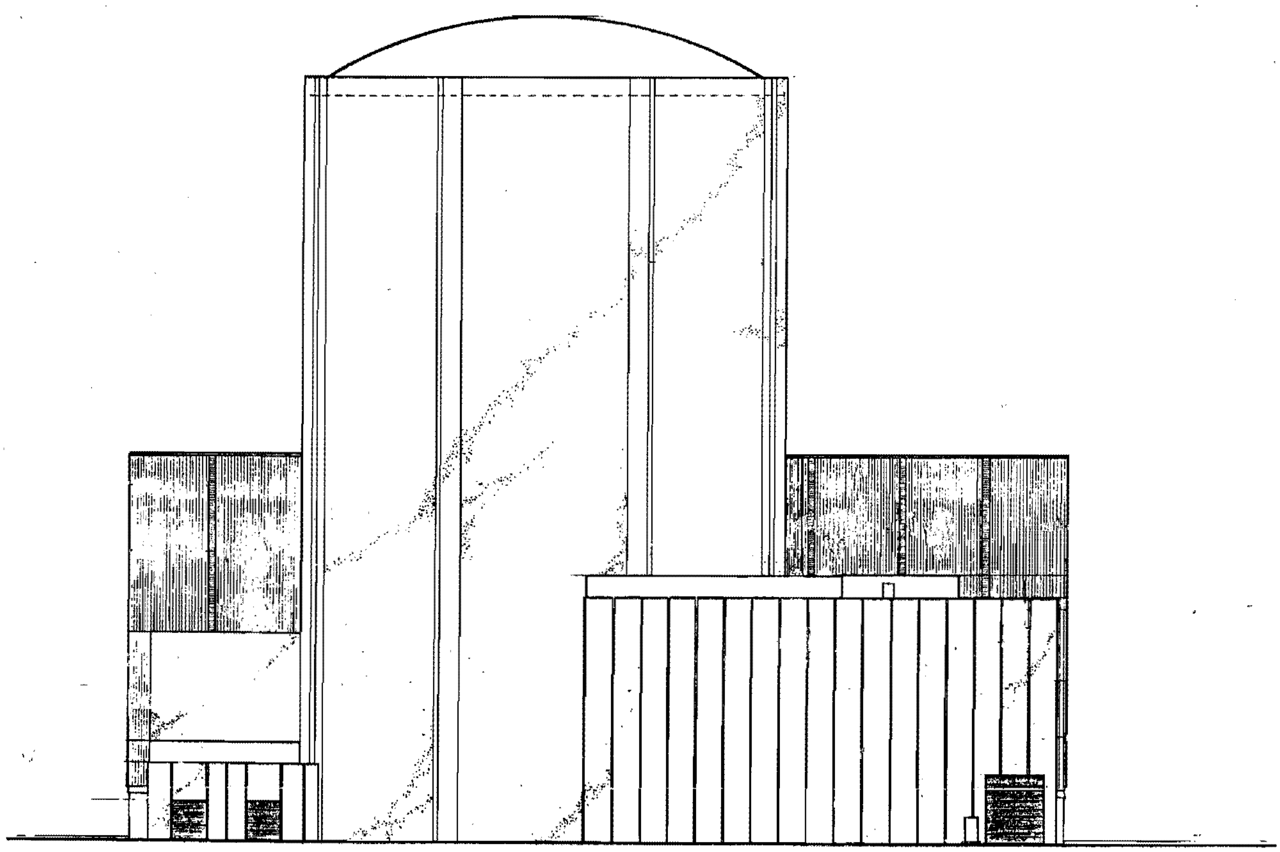
08/10/11	Approval to implement the Engineering Change Package for creation of a temporary access opening through the shield building. [ECP 10-0458]
09/29/11	Approval to commence work on the Order for creation of a temporary access opening through the shield building. [Order 200433294]
10/03/11 @1545	Completed etching the cut line for a temporary access opening through the shield building. [Outage Log]
10/07/11 @0215	Started the diesel engines to commence the hydrodemolition of concrete for a temporary access opening through the shield building. [Outage Log]
10/07/11 @0415	Shutdown the hydrodemolition process to clean out debris hoppers. Approximate depth at six inches in quadrant 1 and four inches in quadrant 3. [Outage Log]
10/07/11 @1700	Resumed the hydrodemolition process through the architectural flute reinforcing steel. [Order 200433294]
10/08/11 @0430	Began reinforcing steel removal from architectural flute. [Order 200460515]
10/08/11 @0730	Inspected reinforcing steel for minimum coverage and identified ¾ inch concrete coverage in some regions of the shield building exterior. [Outage Log & Condition Report 2011-03232]
10/09/11 @1255	Resumed the hydrodemolition process. [Outage Log]
10/09/11 @1700	Continued reinforcing steel removal. [Order 200433294]
10/09/11 @1800	Resumed the hydrodemolition process to expose outer face of structural reinforcing steel on east side. [Order 200433294]
10/09/11 @2210	Continued reinforcing steel removal. [Outage Log]
10/10/11 @0430	Completed the hydrodemolition process phase 2. [Outage Log]
10/10/11 @0450	Continued reinforcing steel removal. [Outage Log]
10/10/11 @1600	Bechtel reported that they were writing a Condition Report for discovery of a concrete crack propagating along the outer face of structural reinforcing steel (#11) across the top and down the side of the shield building temporary access opening. [Outage Log]
10/10/11 @2021	Fractured concrete found at shield building construction opening. During hydrodemolition operations to create the shield building construction opening, fractured concrete was found. The concrete fractures extend into concrete not currently intended to be removed. [Bechtel Condition Report 25539-000-GCA-GAMG-00182 & FENOC Condition Report 2011-03346]
10/10/11	Nuclear Regulatory Commission (NRC) inspectors were immediately notified of the indications of potential fracture lines in the concrete and closely monitored the plant's actions and analysis. [PNO-III-11-014]

3.3 Discussion

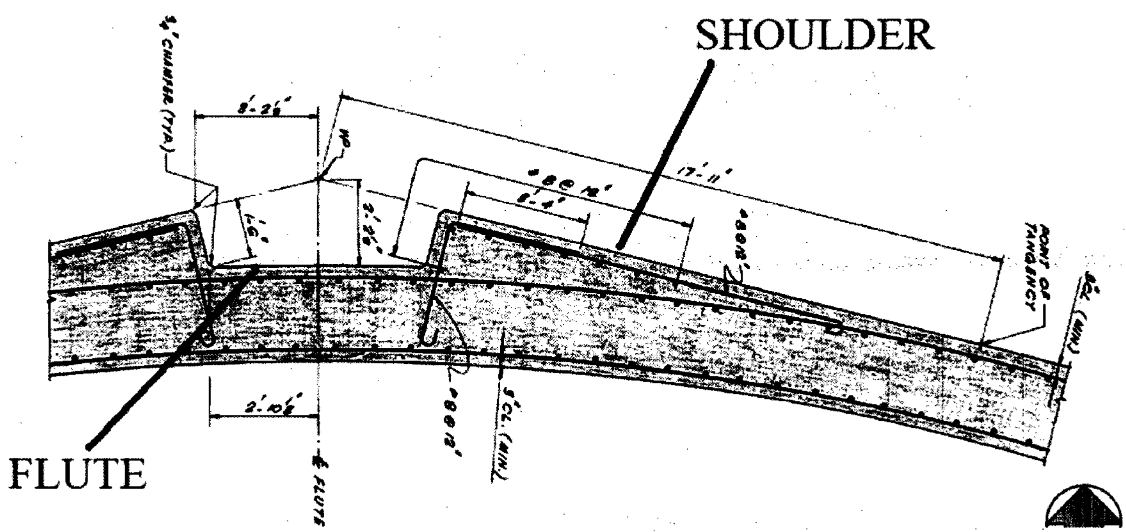
On October 10, 2011, a concrete crack was observed at the architectural flute shoulder region of a temporary access opening in the shield building wall at DBNPS. The temporary access opening was being cut by supplemental personnel under the direction of FENOC using a hydrodemolition process to allow replacement of the reactor pressure vessel head.

The shield building is a reinforced concrete structure of right cylinder configuration with a shallow dome roof [Reference 10.1.1]. An annular space is provided between the steel containment vessel and the interior face of the concrete shield building of approximately 4 feet 6 inches width to permit construction operations and periodic visual inspection of the steel containment vessel. The containment vessel and shield building are supported on a concrete foundation set on a firm rock structure. With the exception of the concrete under the containment vessel, there are no structural ties between the containment vessel and the shield building above the foundation slab.

The shield building has a height of 279 feet 6 inches measured from the top of the foundation ring to the top of the dome and is the tallest building in the power structure. The inside radius of the shield building is 69 feet 6 inches and the thickness of the shield building wall is approximately 2 feet 6 inches. The shield building exterior has eight vertical architectural flute reveals that are spaced 45 degrees apart [Reference 10.1.2]. The architectural flute reveals consist of shoulders that extend another 1 foot 6 inches outward and gradually taper back to the outer cylindrical wall of the shield building while reaching a point of tangency 17 feet 11 inches from the centerline of the flute.



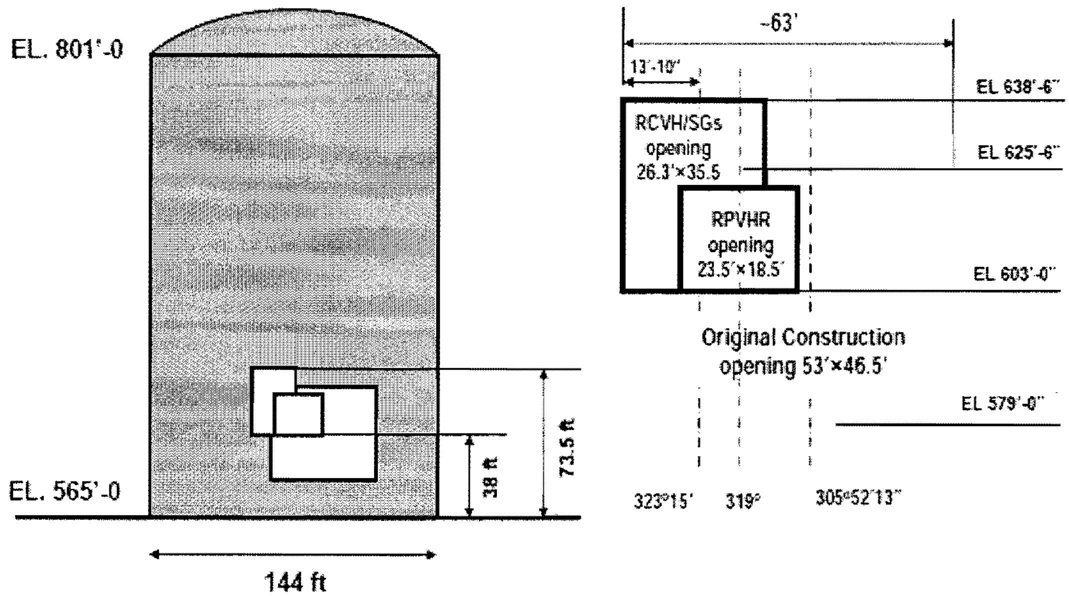
West Elevation View of Plant



Plan View of Architectural Flute / Shoulders

A temporary access opening approximately 25 feet 6 inches wide and 35 feet 6 inches high centered at approximately azimuth 323 degrees was cut through the shield building wall during the 17 Mid-cycle outage at DBNPS [Reference 10.1.3 & 10.1.4]. The left side of the temporary access opening was cut near the thickest section of the architectural flute shoulder located on the north-northwest side of the shield building between elevations 603 feet and 638 feet six inches. The temporary access opening through the shield building wall was necessary for installation of a replacement reactor pressure vessel head. The existing reactor pressure vessel head was installed in 2002 through a temporary access opening located within the boundary of the original construction opening. The current temporary access opening extended past the edge of the original construction opening into part of the previously undisturbed shield building wall.

The shield building temporary access opening was being cut using a hydrodemolition process. Hydrodemolition is a method of concrete removal using high pressure water jets directed towards the surface to separate the course and fine aggregate from the cement paste. Hydrodemolition of concrete was first developed in Europe in the 1970's and has become widely accepted and utilized for concrete removal and surface preparation. Unlike conventional impact methods, hydrodemolition does not result in vibration through the structure or induce micro cracking of the remaining material [Reference 10.1.5]. Hydrodemolition is particularly effective in removing concrete from around reinforcing steel. The temporary access opening made in 2002 for replacement of the original reactor pressure vessel head was also created using a hydrodemolition process.



Elevation View of Shield Building / Construction Opening & Temporary Access Openings

American Hydro personnel performing the hydrodemolition of the shield building temporary access opening noted that towards the left side, between the architectural flute reinforcing steel and the outer face of structural reinforcing steel, the concrete removed consisted of larger chunks that sheared through the aggregate leaving a smooth flat surface. The concrete past (inward of) the outer face of structural reinforcing steel, and also on the right side of the temporary access opening placed during the 2002, was more difficult to remove and the larger chunks were not present [Reference 10.1.6]. The American Hydro personnel observed these conditions while using consistent parameters for the hydrodemolition equipment (water pressure, cut angle, nozzle stand-off, traverse speed, step distance). Therefore, the only variable was determined to be the condition of the shield building concrete.

The known extent of condition at the time of problem discovery was a concrete crack in the shield building wall that was visible on the entire 35 feet height of the temporary access opening left face. The crack was adjacent to the outer face of structural reinforcing steel, and it extended approximately eight feet into the left-top and approximately four feet into the left-bottom region of the temporary access opening progressing from the outer face of structural reinforcing steel towards the architectural flute reinforcing steel. The crack at the left-top of the temporary access opening continued to the exterior surface at the edge of the opening and extended about two and a half feet vertically upwards on the exterior surface of the shield building concrete [Reference 10.1.7]. The crack width was initially indeterminate, due to the effect of the hydrodemolition process, resulting in disintegrating the concrete aggregate and cement paste at the crack location. The hydrodemolition process was incomplete at the time of crack discovery and had been stopped to allow removal of the outer face of structural reinforcing steel.

3.3.1 Initial Problem Solving

FENOC promptly originated a Condition Report and assembled a Problem Solving Team to investigate the unexpected concrete crack within the shield building temporary access opening [Reference 10.1.8]. Members on the Problem Solving Team consisted of Davis-Besse and Beaver Valley engineering staff as well as concrete material experts and structural engineering specialists from Bechtel and Sargent & Lundy to assist with the investigation. The Problem Solving Team on October 12, 2011 performed an initial examination of the concrete crack within the shield building temporary access opening that was confined to the bottom region adjacent to the architectural flute. During this initial examination, the temporary access opening was covered with grit and hydrodemolition residue which obscured the crack details. The Problem Solving Team requested that the temporary access opening be cleaned of the grit and residue for follow-up visual examination to be performed on October 13, 2011.

Excavation of the left side of the temporary access opening commenced with an electric chipping hammer to remove the material influenced by the hydrodemolition process. The chipping revealed that the concrete crack terminated from the bottom left corner to approximately 18 feet above the opening bottom, and a 2 foot section at the top left corner. After examining the region where chipping was performed, the exposed concrete was found to be sound and tightly adhered to the reinforcing steel, and a crack was no longer apparent along the left side of the shield building temporary access opening. Inspection of the reinforcing steel visible at the temporary access opening, and the reinforcing steel already removed from the outer face and architectural flute shoulder region, revealed that the reinforcing steel corrosion was acceptable and there was no evidence of material loss.

Engineering Change Package 10-0458-001C was created to allow excavation beyond the design cut-line in the region at the top-left corner of the temporary access opening. Further chipping at the top-left corner of the temporary access opening resulted in a section excavated upward approximately 23 inches. As a result of the chipping, the overall crack length had been reduced from the initial 8 feet to approximately 5 feet long. The crack in the region at the top-left corner of the shield building temporary access opening remained evident, tight, and confined to the architectural flute shoulder region.

The Problem Solving Team developed an initial work plan to obtain 2 inch diameter core bores to define the extent of a concrete crack at the top-left corner of the temporary access opening in the shield building [Reference 10.1.9]. A core bore approximately five feet above the excavated section at the top-left corner of the temporary access opening was selected to determine if the concrete crack was influenced by either the hydrodemolition process or redistribution of stress adjacent to the opening. If a crack was evident in this original core bore, then the CTL Group would commence Impulse Response testing to further define the crack characteristics followed by additional core bores. The initial core bore was performed and it included the laminar crack identified below.

A grid pattern, 6 feet wide by 15 feet high with one foot intervals, was laid out to perform Impulse Response testing on the shield building exterior oriented about two feet from the edge of the architectural flute and six inches above the temporary access opening. Engineering Change Package 10-0458-001D and Order 200478731 were created to perform five 2-inch diameter core bores along with the corresponding Impulse Response testing.

3.3.2 Impulse Response Testing and Core Bores

Nondestructive examination techniques for concrete have been slow to develop primarily because concrete is an inherently heterogeneous material with constituent materials that can vary widely, depending on geographical region. As a consequence, material in concrete structures tends to be unique and more difficult to characterize nondestructively than other materials.

The Impulse Response testing method [Reference 10.1.10 & 10.1.11] is a direct descendent of methods for evaluating the integrity of concrete piles developed in France in the 1960's. The Impulse Response testing method consists of generating a stress pulse in a structure by a mechanical impact. The force-time function of the impact is monitored using an instrumented hammer to generate the impact, a transducer (geophone) to monitor the dynamic response of the structure to the impulse load, and a computer to acquire, process, and record data. The time records for the hammer force and the geophone velocity response are processed using Fourier transforms of the measured data to obtain the frequency spectra. The resulting velocity spectrum is divided by the force spectrum to obtain a transfer function, referred to as the mobility of the element under test. The test graph of mobility plotted against frequency contains information on the condition and integrity of the concrete tested. When a crack or discontinuity is present within a structural element, the response behavior of the outermost layer controls the Impulse Response test result. The basic theory of dynamic mobility has been utilized for a range of applications such as cracking in concrete and delamination of concrete around steel reinforcement in slabs, walls, and large structures such as dams, chimney stacks, and silos. Impulse Response testing has been demonstrated to detect significant voids and other defects in concrete, and it can be used to survey relatively large areas in a reasonably short time frame.

Experience with Impulse Response testing at Crystal River Unit 3 in 2009 with delamination cracking in the concrete containment structure previously demonstrated that a direct correlation exists between the test results and evidence or absence of a crack in the core bores [Reference 10.1.12]. The Impulse Response testing method was primarily used at the DBNPS to determine whether a crack exists at a given measurement location and for mapping regions of the shield building concrete that contains a crack. The Impulse Response testing method utilized has a limitation such that anomalies existing at depths greater than approximately ten inches may not be detected.

Validation for the Impulse Response testing method consists of taking a core bore from a location where testing shows a concrete crack and then visually confirming the existence of a crack within the bore / core sample.

A numbering scheme was created for identifying the shield building architectural flutes designating them by number one through eight clockwise starting at 22 degrees 30 minutes. A similar scheme was created for identifying the architectural flute shoulders designating them by number one through 16 clockwise starting from the zero degree (North) azimuth. Impulse Response data was taken systematically on a one foot grid pattern starting approximately 24 inches from the architectural flute edge horizontally. Ground penetrating radar was used to define the reinforcing steel layout relative to the intended core bore location. Core bores were labeled based upon the shoulder or flute number, the elevation, and the horizontal grid number. Results from the Impulse Response testing and core bores were compiled into a comprehensive report [Reference 10.1.13].

The core bores were inspected internally using a boroscope to document the depth of the crack, and the crack width was established using an optical crack comparator in accordance with Procedure EN-DP-01512, "Shield Building Concrete Examination."

The results of the Impulse Response test data above the temporary construction opening indicated the presence of a concrete crack vertically within the central portion of the architectural flute shoulder region. The edge of the architectural flute extends approximately 18 inches and indicated no cracking. On October 18, both 2-inch diameter core bores S15-645.5-3 ("A") and S15-653.5-3 ("D") confirmed a concrete crack in the architectural flute shoulder region that extended approximately 7 feet and 15 feet directly above the top-left corner of the shield building temporary access opening. Both of these cores were sent off-site for further independent petrographic examination.

The Impulse Response testing pattern was expanded horizontally to include a grid 10 feet above the entire temporary access opening. Impulse Response testing established that the concrete crack was confined within the architectural flute shoulder 15, as confirmed by core bore S15-646.5-8 ("E") on October 21. Next, increasing the height of the Impulse Response testing pattern vertically to include a grid 40 feet high total and 10 feet wide established the top of the concrete crack as confirmed by core bore S15-674.5-3 ("G"). The concrete crack in shoulder 15 had a laminar orientation located near the outer face of structural reinforcing steel in the architectural flute shoulder region, bounded within an area 8 feet wide by 38 feet above the temporary access opening on the northwest side of the shield building, with a crack width measured at less than 0.010 inches.

The Impulse Response tests were then expanded horizontally to determine if similar conditions existed elsewhere at adjacent architectural flute shoulders 1 and 16 above the emergency diesel generator building roof facing northward and shoulder 13 at the top-right of the original construction opening facing westward. Engineering Change Packages 10-0458-001E, 10-0458-001F and Order 200479708 were created to perform additional 2-inch diameter core bores.

A concrete crack in shoulder 16 and also a crack in shoulder 1 have Impulse Response test data characteristics and orientation similar to the laminar crack originally identified in shoulder 15, on a substantially smaller scale. Condition Report 2011-04214 was originated on October 24, for the discovery of an additional concrete crack in the core bore taken from the architectural flute region of shoulder 16. Core bores S16-613.0-46 and S16-613.0-42 confirmed that the concrete crack in shoulder 16 had a laminar orientation located near the outer face of structural reinforcing steel in the architectural flute shoulder region with a width measured at less than 0.010 inches.

Cracking was identified in shoulder 13 that had similar Impulse Response test data characteristics and orientation to the laminar crack originally identified in shoulder 15. Core bores S13-633.08 and S13-633.0-11 established that the concrete crack terminated prior to the tie-ins and lateral hooks at each end of the architectural flute reinforcing steel. The termination of the concrete crack in shoulder 13 was consistent with the termination of the crack along the left side of the temporary access opening in shoulder 15 that was previously confirmed by chipping back the concrete.

Impulse Response testing and cores bores taken using man-lifts from the ground and scaffold from building roofs across 15 of the 16 architectural flute shoulders confirmed that a similar concrete crack phenomenon in the architectural flute shoulders exists in other regions around the perimeter of the shield building. Examination of nine core bores (S3-650.0-9, S4-650.0-16, S5-666.0-8, S6-666.0-42, S6-666.0-44, S7-666.0-7, S8-666.0-41, S10-666.0-40, and S12-666.0-4) revealed that each concrete crack had a laminar orientation located near the outer face of structural reinforcing steel in the architectural flute shoulder region. Therefore, the shield building concrete crack phenomena were determined to be unrelated to hydrodemolition of the temporary access opening. Shoulder 14 was not accessible from the ground due to interference with a start-up transformer.

A similar concrete crack also was detected in regions without architectural flutes between shoulders 6 & 7 and shoulders 10 & 11, unlike previous examples of the crack phenomena. Condition Report 2011-04402 was originated on October 26, for a concrete crack adjacent to one side of each main steam line penetration blockout at shoulders 6 & 7, and 10 & 11. Five core bores (S7-656.0-6.5, S7-667.0-25, S9-653.0-11, S11-663.75-30, and S11-669.0-17) confirmed that, adjacent to one side of each main steam line penetration blockout through the shield building, there was a concrete crack with a laminar orientation located near the outer face of structural reinforcing steel. Impulse Response testing of a similar size blockout for a containment purge line located one floor below the main steam line penetration blockout determined there was no similar pattern of concrete cracking. Several cracked and un-cracked cores were sent off-site for further independent examination.

The Impulse Response testing data of the shield building also indicated that the concrete crack phenomena continued upward beyond the reach of the available scaffold and man-lifts. Therefore, southward facing shoulder 9 was selected to determine the vertical extent of the concrete crack while using a man-basket suspended from the shield building parapet. The Impulse Response test data for shoulder 9 indicated three cracked sections that encompassed the majority of the vertical distance of the architectural flute region from the roof of the auxiliary building near elevation 665 feet to the top of the shield building near elevation 795 feet. The concrete crack in shoulder 9 had similar Impulse Response test data characteristics and similar crack orientation originally identified in shoulder 15. Core bores S9-666.0-11, S9-680-3, and S9-785.0-22.5 confirmed that the concrete crack in shoulder 9 had a laminar orientation located near the outer face of structural reinforcing steel in the architectural flute shoulder region with a crack width measured at less than 0.010 inches.

The concrete crack near the top 20 feet of the shield building also continued into the region without architectural flutes between shoulders 8 & 9. Condition Report 2011-04648 was originated on October 31, for a concrete crack outside the architectural flute shoulders located within elevation 780 feet and 801 feet between shoulders 8 & 9. Additional Impulse Response test data identified similar situations where the concrete crack continued into the region without architectural flutes between shoulders 10 &11 and shoulders 12 & 13. Core bores S9-785.0-22.5, S10-790.5-25, and S2-798.5-4.5 confirmed that the concrete crack, which continued into the region without architectural flutes near the top of the shield building, had a laminar orientation located near the outer face of structural reinforcing steel with a crack width measured at less than 0.010 inches. Impulse Response testing of the spring line at the top 5 feet indicated a lesser extent of concrete cracking than the remainder of the top 20 feet of the shield building.

Condition Report 2011-05475 was originated on November 16, 2011 for additional assessment of the concrete cracking in the shield building wall above elevation 780 feet. Impulse Response testing also identified a concrete crack across the approximate 5 foot 8 inch region between both architectural flute shoulders 5 and 6 at the top of the shield building. Core bore F5-791.0-4 confirmed that the concrete crack across the architectural flute near the top of the shield building had a laminar orientation located near the outer face of structural reinforcing steel with a crack width measured at 0.013 inches.

Eight core bores were drilled deep into the shield building to ensure that the cracked concrete was confined to the outer face of structural reinforcing steel. Five core bores (S7-782.0-8.5, F2-790.0-4.5, F4-794.0-3.5, F4-791.0-2.5, and F5-791.0-4) confirmed that there was no similar concrete cracking inward from the outer face of structural reinforcing steel near the top of the shield building wall. Three other core bores (S16-613.0-30, S16-613.0-42, and F7-633.08) confirmed that there was no similar concrete cracking inward from the outer face of structural reinforcing steel in the lower half of the shield building wall. Additionally, visual examination of the entire perimeter of the temporary access opening did not reveal any laminar cracking at the inside face of structural reinforcing steel.

Evidence of subsurface cracking, other than a laminar crack in the shield building concrete, was also identified on five core bores. Longitudinal cracks, attributed to concrete shrinkage, were discovered in core bores F7-633.08 and F2-790.0-4.5 as described in Condition Reports 2011-04507 and 2011-05648. Longitudinal cracks of the material extracted from core bores F4-794.0-3.5, and F5-791.0-4 also were seen. Another imperfection located approximately one inch below the surface was discovered in core bore S10-672.0-34 as described in Condition Report 2011-04507. Each of these five cores, with indications other than laminar cracking in the shield building concrete, were sent off-site for further independent examination.

Four 3-inch and six 4-inch diameter core bores were collected to support destructive examination and the confirmation of material properties for the shield building concrete, in addition to samples from the 2-inch diameter cores collected while confirming crack locations. Engineering Change Packages 10-0458-001H and 10-0458-001L Order 200478731 were created to perform the 3-inch and 4-inch diameter core bores to support destructive examination of the shield building concrete. The first 3-inch diameter core bore was extracted from the passage to the emergency diesel generator rooms to provide a baseline sample that was not exposed to environmental factors experienced by the shield building exterior walls. The second 3-inch diameter core bore was extracted from the main steam line area to provide a sample that experiences the worst chronic thermal factors. Two other 3-inch diameter core bores were extracted from the northeast and southern facing sides, at approximately mid-span, to provide samples exposed to a differing magnitude of thermal factors experienced by the shield building exterior walls. Six 4-inch core bores were extracted to perform tensile, creep, freeze / thaw, and moisture testing of the shield building concrete.

Engineering Change Package 10-0458-001K was created to provide a sketch / drawing [Reference 10.1.14] showing locations of core bores. The shield building Impulse Response average mobility values are provided in Attachment 1, and the shield building core bore summary is provided in Attachment 2.

Conclusion from Impulse Response Testing and Core Bores

Core bore sample results confirmed that the shield building walls contained a concrete crack that had a laminar orientation located near the outer face of the structural reinforcing steel. The crack widths were found to be generally tight, less than or equal to 0.010 inches, with one crack measuring 0.013 inches. Inspection of the concrete cores indicates that the crack passed through the coarse concrete aggregate.

Eight deep core bores were taken to confirm that the laminar cracking is only located near the outer face of the structural reinforcing steel. In addition, a thorough visual inspection of the entire perimeter of the temporary access opening revealed no laminar cracking at the inside face of the structural reinforcing steel.

Fifteen of the sixteen shoulders were inspected using the Impulse Response mapping. Results indicate that all 15 shoulders inspected had indications of laminar cracking. The cracks were well confined in the shoulder areas between the ends of the horizontal reinforcing steel. The southwestern exposure of the shield building wall was observed to have the most extensive concrete cracking.

Impulse Response mapping identified a region of laminar cracking adjacent to the shoulder areas near the top 20 feet of the shield building. This area coincides with a higher density of reinforcing steel (#11 horizontal bars at 6" c-c).

Impulse Response mapping also identified a region of laminar cracking adjacent to the main steam line construction blockout for penetration #39 and #40, located above and below the Auxiliary Building roof line. This crack is also located in an area of higher density horizontal reinforcing steel.

As an extent of condition associated with the main steam line penetration line blockouts, Impulse Response mapping was conducted adjacent to a similar construction blockout (the containment purge outlet blockout for penetration #34) that also has a higher density of reinforcing steel adjacent to this construction opening. This blockout is located one floor below the main steam line penetration blockout. Impulse Response mapping in this area indicated no laminar cracking. This was the expected result considering this shield building penetration was not exposed to exterior environmental conditions, nor located near any of the architectural flute shoulders.

3.3.3 Petrographic and Destructive Examination

The root cause team visually conducted examinations of a number of larger pieces of concrete removed from the shield building temporary access opening. These samples of concrete debris had fracture surfaces that propagated directly through the large and small aggregate indicating that the bond between the aggregate and the cement paste was very strong. Subsequent visual examination of the concrete cores also found similar transverse fractures through the aggregate.

Thirty-six of the 70 cores extracted from the shield building concrete were subjected to either petrographic or destructive examination by external laboratories in order to quantify material properties for modeling and analysis, or to define possible failure modes. Some of the shield building concrete cores were sectioned and subjected to more than one form of examination. A shield building core bore summary is provided in Attachment 3.

Petrographic Examination

Four of the 36 shield building concrete cores were submitted to two separate laboratories for petrographic examination per ASTM C856, "Standard Practice for Petrographic Examination of Hardened Concrete." Petrographic examination of the mix design and the material properties of concrete were performed in order to determine the condition, possible reasons for damage, and prediction of whether deterioration may continue. Petrographic examinations with a Scanning Electron Microscope are used to determine the physical and chemical characteristics of the concrete. This includes but not limited to; the relative amounts of constituents of the sample, presence of unstable minerals such as soluble sulfates, microcracks, and chemical or physical deterioration. The four cores submitted for petrographic examination of the shield building concrete included the initial 2-inch diameter cores bores collected (S15-645.5-3 & S15-653.5-3), and two of the 3-inch diameter cores (F2-792.3-4.5 & F4-791.0-2.5). A point-count analysis was also performed of core F4-791.0-2.5 per ASTM C457, "Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete."

The CTL Group petrographic examination [Reference 10.1.15] of cores S15-645.5-3 ("A") and S15-653.5-3 ("D") determined that the shield building concrete is in very good condition and that the limited carbonation at the fracture surface indicates that the laminar crack was not directly exposed to air. Transverse cracks in both cores, associated with those crack locations identified in the core holes, pass through coarse aggregate. Fracture surfaces were clean, with no discoloration or debris and few deposits. While the concrete represented by the cores exhibited some non-uniformity, the concrete was consistent with the mix designs provided in terms of composition and quality.

The Wiss, Janney, Elstner Associates petrographic examination [Reference 10.1.16, Exhibit 26] of cores F2-792.3-4.5, and F4-791.0-2.5 determined that the concrete generally corresponds to the mix design with moderate variability with respect to its consolidation, air-void system, and water-to-cement ratio. The specific surface and spacing factors of the air-void system in core F4-791.0-2.5 calculated by the point-count analysis did not meet industry requirements for freeze / thaw durability in moist conditions, but because of the increased strength, no evidence of freeze / thaw deterioration was detected in either core F2-792.3-4.5 or F4-791.0-2.5.

The two separate petrographic examinations determined an estimated air content that ranged from 1 to 3 percent in core S15-645.5-3, from 1 to 3 percent at the outer end and 3 to 5 percent in the body of the core S15-653.5-3, and 5 percent in core F4-791.0-2.5. The concrete was air-entrained, but some air contents were lower than the 4.5 to 6 percent specified in the original mix designs. The water-to-cement ratio for cores S15-645.5-3 and S15-653.5-3 was estimated between 0.45 and 0.55, and 0.38 to 0.42 for core F2-792.3-4.5. Limited petrographic examination of core F4-791.0-2.5 identified a variable water-cement ratio similar to core F2-792.3-4.5, but to a lesser extent. Core F2-792.3-4.5 had moderate amounts of thin, soft paste zones with an elevated water-to-cement ratio and air voids which represent areas of weakness and increased potential for fluid penetration.

The outer surface of the cores was covered with a mortar coating of variable thickness that was not water-repellant, and the paste was highly absorbent along the outer surface. Cores F2-792.3-4.5 and F4-791.0-2.5 contained some larger and abundant medium sized air voids near the surface indicating less than optimal consolidation. The air voids of core F2-792.3-4.5 and F4-791.0-2.5 were lined with secondary deposits of ettringite (crystal formation from sulfate reaction with calcium aluminates) and calcium hydroxide which suggests long-term exposure to moisture migrating through the concrete. The near surface zone of core F2-792.3-4.5 was considered to be relatively poor. The paste along the outer surface (shield building exterior wall) of cores S15-645.5-3 and S15-653.5-3 was fully carbonated to a depth of 0.2 to 0.3 inches, and typically 0.25 inches for core F2-792.3-4.5. These amounts of carbonation are typical for a concrete surface exposed for 40 years.

The paste along the outer surface of cores S15-645.5-3 and S15-653.5-3 (exterior surface of the Shield Building wall) is fully carbonated to a depth of 5 to 8 mm. Carbonation in the body of the cores exhibits a mottled pattern with small areas of carbonated and non-carbonated paste; however, this feature does not appear to affect the overall integrity and performance of the concrete. Paste along the fracture surfaces of both cores, associated with those crack locations identified in the core holes, exhibits the same mottled carbonation pattern observed in the body of the cores; however, the paste does not appear to have carbonated due to exposure along the fracture surfaces.

The water soluble chloride in cores S15-645.5-3 and S15-653.5-3 ranged between 310 and 370 parts per million. No materials-related causes for the cracks and microcracks were observed, and no evidence of chemical reactions involving aggregates and paste constituents (such as alkali aggregate reaction) was observed.

<u>Destructive Examination</u>

Thirty-three of the 36 shield building concrete cores were submitted to four other laboratories for destructive examination in order to quantify material properties for modeling and analysis, or to define possible failure modes. The four locations examining the shield building concrete cores were the United States Bureau of Reclamation (USBR), the University of Colorado – Boulder, the Twining Laboratory, and the PhotoMetrics laboratory. A listing of the shield building concrete material property tests performed for each core are provided in Attachment 3.

Two shield building 3-inch diameter concrete cores (F4-791.0-2.5 and S7-782.0-8.5) were tested by the USBR for the material properties of thermal diffusivity, specific heat, thermal conductivity (calculated), and coefficient of linear thermal expansion [Reference 10.1.16, Exhibit 59]. Thermal diffusivity is the rate at which temperature changes take place in concrete and is measured per test procedure USBR 4909-92. Specific heat is the rate at which heat is transmitted through a unit thickness of the concrete and is measured per test procedure USBR 4907-92. Thermal conductivity is the rate at which heat is transmitted through a unit thickness of material and is calculated from the thermal diffusivity, specific heat, and density of the concrete. The coefficient of linear thermal expansion is the change in unit length per degree of temperature change of the concrete and is measured per test procedure USBR 4910-92.

Two shield building 4-inch diameter concrete cores (S1-615-2 and S3-650-2) were tested by the Twining Laboratory for the material properties of modulus of elasticity, compressive strength, and splitting tensile strength [Reference 10.1.16, Exhibit 3]. The modulus of elasticity provides the stress to strain ratio and a ratio of lateral to longitudinal strain in hardened concrete and is measured per ASTM C469. The compressive strength provides a basis for determination of compliance with concrete proportioning, mixing, and placing specifications and is measured per ASTM C39. The splitting tensile strength evaluates the shear resistance of concrete and is measured per ASTM C496.

Eight shield building concrete cores were tested by the University of Colorado - Boulder consisting of three 3-inch diameter cores (S9-680-3, EDG passage, and Main Steam line room), and five 4-inch diameter cores (S2-616-14, S3-650-2, S4-649-22, S6-665-47, and S8-665). The material properties tested by the University of Colorado – Boulder were internal relative humidity, compressive strength, splitting tensile strength, coefficient of thermal expansion, and accelerated creep [Reference 10.1.16, Exhibit 60]. A freeze / thaw resistance test per ASTM C666 was aborted due to test equipment malfunction which inhibited completion of the test within the time available. Petrographic examination had already shown no evidence of freeze / thaw deterioration. The internal relative humidity is important for evaluating potential shrinkage and freeze / thaw damage in concrete. The accelerated creep test measures the load-induced time-dependent compressive strain of the concrete per ASTM C512.

None of the material properties for the shield building concrete were found to be unacceptable relative to typical results from the test standards. The specific heat and thermal diffusivity of the shield building concrete were greater than typical ranges of thermal properties for normal concrete, but the thermal conductivity calculated from these values was within the typical range for normal concrete. The coefficient for thermal expansion and concrete creep were also within the typical range for hardened concrete. The internal relative humidity test result was typical of the annual value in the environment. The compressive strength was greater than minimum design value, and the modulus of elasticity test result was greater than the calculated design value. The splitting tensile strength was nearly double the value calculated from the design compressive strength. A tensile strength of 600 pounds per square inch was established as a material property for the shield building concrete based upon 10 percent of the average 28-day compressive strength (6000 pounds per square inch) as shown in Attachment 8. The examinations found no evidence of chronic thermal factors in the concrete cores collected at the various diverse orientations of the shield building wall.

Twenty-two of the shield building concrete cores were examined by PhotoMetrics Laboratories using a scanning electron microscope for characteristics such as aggregate size, void fraction, concrete-to-reinforcing steel interaction, carbonation, and fracture analysis.

A methodology similar to ASTM C457 determined the void fraction for the concrete cross-sections and areas with reinforcing steel contact were not substantially different. Two core samples with reinforcing steel interaction had iron oxide transfer to the concrete and one had an imprint from the reinforcing steel deformation ribs.

The carbonation measured on the exterior surface of the cores was consistent with the depth measured by the petrographic examination from other laboratories. The carbonation measured on both transverse and longitudinal crack surfaces was minimal (average 0.62 millimeters) and inconsequential. These trace amounts of carbonation do not adversely affect the reinforcing steel or the capacity of the structure. The fracture analysis found no evidence of microcracks with magnifications up to 500 times on virtually all the samples.

Reinforcing steel samples taken from the shield building temporary access opening were also examined by PhotoMetrics. These reinforcing steel samples did not exhibit excessive corrosion or material loss and were representative of the minor amount of corrosion expected during staging for the shield building construction. There was no evidence of extensive corrosion or deterioration that would be expected if the reinforcing steel had been exposed to moisture during plant operation after the shield building had been constructed.

Conclusion from Petrographic and Destructive Examination

The external laboratory examination of the shield building concrete core samples determined that the concrete was in good condition and consistent with the mix design. One core did not meet industry requirements for freeze / thaw durability, however the higher apparent compressive strength of the concrete provides resistance which is the key component to freeze / thaw deterioration. Long term exposure to moisture is clearly evident in the core bores.

General properties of concrete that may promote cracking include, by are not limited to, high volume of paste, elevated water-cement ratio and unsound aggregate. The overall volume of paste in the DBNPS concrete is not considered to be high. The overall estimated water-cement ratio of the concrete is considered to be low. The crushed limestone aggregate appears to be chemically and physically sound. In general, variability in concrete may lead indirectly to cracking if areas of poor consolidation, elevated water-cement ratio, or concentration of air void are extensive; however, this does not appear to be the case in the concrete represented by the cores removed from the shield building.

The exposed concrete surface had carbonation typical for a concrete structure of 40 years. In addition the thickness of the concrete cover is much larger (3-inches) than the carbonation depth (<1/2-inch) and thus the current carbonation does not reduce the concrete's ability to protect the reinforcing steel.

The interior cracked concrete surfaces had trace amounts to no indication of carbonation which indicates that the cracked surfaces do not affect the concrete's ability to protect the reinforcing steel.

The lack of microcracks on the fracture surfaces eliminates a progressive aging failure or fatigue mechanism. There was no evidence of typical concrete time-dependent aging failure modes such as chemical attack including reinforcing steel corrosion, physical attack, chronic freeze / thaw, and vibration / fatigue. The integrity of the concrete narrowed the failure mechanism to those related to design or environmental issues versus construction. The observed extent of laminar cracking in the shield building together with the core examination results indicated that the mechanism was most likely caused by an acute and large radial force as opposed to gradual degradation over time due to smaller cyclical forces.

The examination found the outer surface of the cores was not water-repellant, and the air voids were lined with secondary deposits of ettringite and calcium hydroxide which suggests long-term exposure to moisture migrating through the concrete.

The physical properties of the concrete were obtained and used as input to shield building modeling and analysis described later in this report.

3.3.4 Surface Examination

A surveillance test and a maintenance rule structural inspection procedure govern the periodic surface visual inspection of the shield building at DBNPS. Procedure DB-PF-03009, Containment Vessel and Shield Building Visual Inspection, satisfies the surveillance requirement of Technical Specification 3.6.1.1 for performing the required visual examination of the shield building in accordance with the Containment Leakage Rate Testing program, and also for maintenance or modification testing. Personnel who perform the shield building visual inspection via procedure DB-PF-03009 meet the requirements for a general visual examiner. The accessible interior and exterior surfaces of the shield building are examined for evidence of flaking, spalling, discoloration, voids, cracks, or other signs of distress. Any conditions that may affect structural integrity, or otherwise not meet the acceptance criteria, are documented on a Condition Report. Other insignificant scratches, dings, chips, or abrasions may at the discretion of the inspector be documented on a Notification.

Procedure EN-DP-01511, Design Guidelines for Maintenance Rule Evaluation of Structures, satisfies the criteria for evaluation of safety-related structures that are relied upon to mitigate an accident or transient or whose failure could prevent safety-related structures, systems or components from performing their safety-related function. Personnel who perform the shield building visual inspection via procedure EN-DP-01511 are degreed engineers with a minimum of 5 years experience in civil structural engineering activities, with the ability to judge deficiencies potentially affecting the structural adequacy of the respective structures. Concrete cracks less than 1/16 inch width need not be evaluated unless they have developed through the entire thickness. Any conditions that impair the structure capable of performing its intended function, or could deteriorate to an unacceptable condition are identified on a Condition Report.

A shield building surface visual inspection history is provided in Attachment 3. The Maintenance Rule Structure Evaluation from June 1999 and November 2005 identified surface cracks, but since they were all less than 1/16 inch, the cracks were found to be acceptable. Since May 1996, the surface visual inspections of the shield building exterior have identified concrete spalling above the original construction opening. The concrete spalling above the original construction opening coincides with the location of the various grout tubes used for closing the blockout as shown on drawing C-112 detail 1. Previous inspections of the shield building exterior surface did not identify symptoms that would signify the presence of the concrete laminar cracking.

Three other Condition Reports initiated during the 2011 mid-cycle outage describe localized concrete surface distress consisting of surface cracks, spalling, and exposed reinforcing steel on the shield building. These three conditions were discovered subsequent to the original crack at the left-top of the temporary access opening which continued to the exterior surface. The other concrete surface distress conditions were evaluated separately from this root cause analysis and determined to not impact the structural integrity of the shield building.

Condition Report 2011-04190 was originated on October 23, 2011 for several instances observed from the roofs of the auxiliary building and emergency diesel generator rooms of concrete surface cracking on the shield building exterior. The shield building exterior surface concrete cracks varied in length, and were tight with the greatest width measured at 0.025 inches. The surface cracks are not a structural concern due to their tightness.

Condition Report 2011-04507 was originated on October 28, 2011 and describes several minor tight concrete surface cracks on the shield building exterior near an anomalous indication from Impulse Response testing and core bore S10-672.0-34. These surface cracks were less than three feet in length and had depths of approximately one inch from the surface. Another approximately one square foot area with minor concrete surface cracks was located on shoulder 15 in close proximity to a second anomalous indication identified by Impulse Response testing. There was no spalling or signs of staining associated with either of these locations having concrete surface cracks on the shield building exterior. The surface cracks were determined to not be a structural concern due to their tightness.

Condition Report 2011-05648 was originated on November 18, 2011 for exposed reinforcing steel and spalling on the shoulder 4 corner located above the emergency diesel generator room roof, and a horizontal shrinkage crack extending from flute 2 to the face of shoulder 4 at approximately 797 feet elevation. The spalling condition on shoulder 4 originated from reinforcing steel which was exposed to the environment and subjected to localized corrosion. This exposed reinforcing steel and associated spalling is confined to the corner of architectural shoulder 4, and therefore is not of a structural concern nor will it impact the ability of the Shield Building to perform its design function.

After returning DBNPS to service in December 2011, thermal imaging was performed at the concrete near the main steam line penetration blockouts of the shield building with the unit at 100 percent power. The thermal imaging determined that concrete above main steam line penetration 39 is about 107 degrees Fahrenheit, and about 101 degrees Fahrenheit at the adjacent core bores (S7-652.0-6.5 & S7-652.0-6.5). The thermal imaging determined that concrete above main steam line penetration 40 is about 122 degrees Fahrenheit, and about 108 degrees Fahrenheit at the adjacent core bores (S9-650.0-9 & S9-653.0-11). These readings on the surface of the concrete at the main steam line penetration blockouts of the shield building are substantially less than the general 150 degree or localized (penetration) 200 degree Fahrenheit temperature recognized as the threshold for a reduction of strength and modulus due to elevated temperatures. Additional description of the potential impact from a high temperature environment is provided in Attachment 12 (failure mode 3.7).

Conclusion from Surface Examination

Various visual inspections of the shield building exterior surface performed over many years did not identify any unusual surface defects or symptoms of distress that would signify the presence of the subsurface concrete laminar cracking. The items identified were localized surface conditions unrelated to the shield building concrete laminar cracking.

As described above, the shield building is inspected by visual examination of the concrete surface. This inspection is specifically reviewing the structure for cracks, concrete spalling, and scaling (loss of cement and fine aggregate at the concrete surface). These may be indications of a structural concern requiring further investigation.

This method of inspection, by definition, can not detect subsurface cracks. The visual method of inspecting concrete structures is the standard for nuclear power plant structures. The corrective actions section of this report detail the DBNPS plan for the long term monitoring of the shield building laminar cracks.

3.3.5 Design

The original concept for DBNPS included a post-tensioned concrete containment with steel liner similar to other plants with a nuclear steam supply system from Babcock & Wilcox. A post-tensioned concrete containment consists of steel wire tendons that are installed, tensioned, and then anchored to the hardened concrete forming the structure. The tendons counteract tensile loads by subjecting the concrete to high compressive forces to prevent or minimize cracking and also improve resistance to shear forces that could develop during accident conditions.

In April and May 1969, the Chicago Bridge and Iron Company submitted proposals for a free-standing steel containment vessel with a reinforced concrete shield building at DBNPS. The Chicago Bridge and Iron Company would subcontract for construction of the shield building, but the design would be the responsibility of the Bechtel Corporation. On May 22, 1969 the Toledo Edison Company instructed Bechtel to proceed with a containment system for the station utilizing a free-standing containment vessel surrounded by a reinforced concrete shield building instead of the post-tensioned concrete containment with steel liner.

The shield building is designed to provide biological shielding during normal operation and from hypothetical accident conditions [Reference 10.1.1]. The shield building provides radiation shielding, a means for collection and filtration of fission product leakage from the containment vessel following a hypothetical accident, and environmental protection for the containment vessel from adverse atmospheric conditions including extreme winds, tornadoes, and tornado-borne missiles. Besides the emergency ventilation system, the shield building also interfaces with station lightning protection, and station drainage.

The shield building is a structure consisting of concrete, and reinforcing steel, with other minimal miscellaneous embedded material. The shield building was designed in accordance with American Concrete Institute (ACI) 307-69, Specification for the Design and Construction of Reinforced Concrete Chimneys, and checked by the Ultimate Strength Design Method in accordance with ACI 318-63, Building Code Requirements for Reinforced Concrete.

The design of the shield building considered the structure dead load, live load, earthquake load, wind load, tornado load, external missiles, thermal load, and a loss of coolant accident. The dead load considered the density of the shield building concrete and reinforcing steel. The live load considered 40 pounds per square foot on the shield building dome. The earthquake load considered a horizontal ground acceleration of 0.15g acting concurrently with a vertical ground acceleration of 0.10g.

The wind load considered 90 miles per hour at 30 feet above grade which represents the highest wind speed expected for a hundred year period. The tornado load considered a pressure drop of 3 pounds per square inch in 3 seconds which represents twice the greatest pressure drop ever reliably measured. The tornado load also considered a lateral force based upon a funnel with a tangential velocity of 300 miles per hour. Objects of low cross-sectional density such as boards, metal siding, and similar items may be picked-up and carried at the maximum wind velocity of 300 miles per hour. The external missiles considered include a 12 foot long piece of wood 8 inches in diameter traveling on end at a speed of 250 miles per hour, a 4000 pound automobile traveling through the air at 50 miles per hour not more than 25 feet above ground, and a 10 foot long piece of pipe of 3.4 inches outside diameter traveling on end at a speed of 100 miles per hour.

The thermal load considered a temperature of negative 17 degrees Fahrenheit in the winter on the outside of the shield building, with an annulus temperature of 110 degrees Fahrenheit. The loss of coolant accident load considered a temperature of negative 17 degrees Fahrenheit in the winter on the outside of the shield building, with an annulus temperature of 152 degrees Fahrenheit. The design basis thermal load calculations for the shield building did not take any structural credit for the mass of the architectural flute.

Load combinations specified in ACI 307-69 provide the design basis of the shield building. The smallest safety margin is greater than the ACI 318 code requirements by 3 percent and located on the vertical reinforcement at the base of the shield building. The smallest safety margin applies to the combination of dead load, live load, earthquake, and thermal load.

The concrete for the shield building is a dense durable mixture of coarse aggregate, fine aggregate, cement, and water. An air-entrainment admixture and a water-reducing & retarding admixture were added to improve the quality and workability of plastic concrete during placement and to retard the set of the concrete. The air-entrainment admixture increases the resistance to freeze / thaw cycles in the concrete by creating air voids which allow water to move to the void during freezing. The water-reducing & retarding admixture were utilized to reduce the shrinkage and creep of the shield building concrete. Type II cement was used below-grade for its greater resistance to aggressive chemical attack from sulfates in the fill material placed against the structure. A waterproofing membrane is used around the below-grade portion of the shield building exterior. No exterior protective sealant was specified as a barrier against moisture migrating into the structure. The concrete for the shield building has a design compressive strength of 4000 pounds per square inch at 28 days for the wall, and wall below grade.

As described above, the shield building wall contains concrete with a minimum compressive strength of 4000 pounds per square inch. Reinforced concrete principles dictate that the concrete carries the compressive loads and the provided reinforcing steel carries the tension loads. This arrangement is due to the inherently low tensile strength of concrete. The tensile strength of concrete is a more variable material property than the compressive strength. The tensile strength typically ranges between only 10 to 15 percent of the compressive strength of the concrete, which is the reason that concrete is not designed to carry tensile loads.

The structural reinforcing steel in the shield building consists of deformed billet steel bars. The reinforcing steel is placed in the concrete walls, dome, and foundation for tensile strength to control cracking due to concrete shrinkage and temperature gradients. The design of the shield building ensures an elastic behavior of the reinforcing steel during a maximum possible earthquake which controls cracking of concrete and impairment of leak tight integrity. Major openings and penetrations through the shield building are designed such that, the anticipated loads are carried by frame action around the openings. This frame action is achieved by adding sufficient reinforcement around the perimeter of the openings and adding diagonal bars at each corner of the openings to provide tension and shear capacity. Corrosion protection for the reinforcing steel in the shield building is provided by a minimum of two inches of concrete cover at either face of the wall.

The radial cracking effect of reinforced concrete is considered for thermal load as suggested by ACI 307-69. This cracking is limited by using a relatively large amount of reinforcing steel. Based upon ACI publication SP-20, "Causes Mechanism and Control of Cracking in Concrete," under the operating conditions the maximum width of cracks is 0.009 inch, which is less than the allowable (0.01 inch) permitted by the ACI Standard Building Code.

The primary pattern for the outside face structural reinforcing steel is #11 at 12-inch on-center in the horizontal and vertical direction as shown on Drawing C-110. Exceptions to this reinforcement pattern exist in areas of increased loading such as sections below grade, at the top 20 feet / spring line, and at openings or penetrations. The architectural flute shoulder horizontal reinforcing steel is #8 at 12-inch centers. The architectural flute shoulder has radial reinforcing steel that interfaces with the outer face of structural reinforcing steel adjacent to the tangential end, and interfaces with the inner face of structural reinforcing steel at the thickest part of the shoulder. No other tie-bars or stirrups connect the outer face of structural reinforcing steel to the architectural flute shoulder region besides that at the tangential end and the thickest part of the shoulder.

The original design of the shield building is robust as demonstrated in the following discussion of the calculations performed to evaluate the laminar cracking observed.

The assessment of the concrete cracking resulted in the development and approval of three new calculations to establish reasonable assurance that the shield building can perform its intended design functions [Reference 10.1.19]. In addition to establishing reasonable assurance that the shield building can perform its intended design functions, these calculations also identify several areas of additional margin where credit is not taken such as with tornado design and damping criteria, and actual strength of reinforcing steel and concrete tested as greater than the minimum design basis. These calculations are not credited as design basis analysis as they were performed to establish reasonable assurance that the shield building can perform its intended design functions, and not performed to support full licensing compliance with all design basis loads, load combinations, and allowable stress requirements. **Direct Cause Corrective Action #2** re-establishes design and licensing basis conformance for the shield building with the observed concrete cracking.

The first new calculation [Reference 10.1.20] analyzed the shield building with the vertical outer face of structural reinforcing steel considered ineffective in the sixteen architectural flute shoulder regions and adjacent to the main steam line penetration blockouts based on the extent of concrete cracking found by impulse response testing and core bores. This analysis evaluated the building for the applicable load combinations described in USAR Section 3.8.2.2. This analysis evaluated the structure considering the reinforcing steel discussed above as being ineffective for the accident loading combinations. The analysis evaluates the shield building using several approaches to ensure that the overall structure remains adequate with the ineffective reinforcing described above. The calculation concludes that the limiting design condition for the vertical reinforcement occurs towards the top of the structure. The analysis concluded that the structure has a demand to capacity ratio of 0.90 which is less than the limit of 1.0. This calculation also identifies additional conservatisms that were not explicitly used in the evaluation. These conservatisms include potential use of the revised tornado design criteria of Regulatory Guide 1.76, Rev. 1, increased seismic damping ratio of Regulatory Guide 1.61, Rev. 1, actual material strengths as documented in test reports. This analysis concluded that the shield building, as evaluated, remains adequate to perform its design basis functions.

The second new calculation [Reference 10.1.21] analyzed the shield building reinforcing steel to determine if the sixteen architectural flute shoulder regions remain adequately anchored for seismic conditions. The analysis considers the reinforcing steel to carry all loads due to the identified cracking in the architectural flute shoulder area for the dead and safe shutdown earthquake (SSE) load combination. This analysis concluded that the architectural flute shoulder regions will remain anchored by the reinforcement to the shield building and do not present a seismic II/I or missile hazard.

The third new calculation [Reference 10.1.22] analyzed the horizontal outer face of structural reinforcing steel based on the extent of concrete cracking found by impulse response testing and core bores. This analysis calculated the hoop stresses at the critical locations of the spring line area between elevations 780 & 800 feet, the main steam line penetration blockout areas, and at a critical location on the architectural flute shoulder region. This limiting portion of this analysis is the main steam line penetration blockout area where two-thirds of the horizontal hoop reinforcement is considered to be ineffective. The analysis has determined that the stresses in the remaining horizontal hoop reinforcement are approximately 73% of the allowable stress limit. This analysis did consider the reduced tornado differential pressure loading of Regulatory Guide 1.76, Rev. 1 to calculate this reinforcement stress ratio. The use of the regulatory guidance is acceptable for this functionality review type of calculation. The calculation concludes that the shield building, as evaluated, remains adequate to perform its design basis functions.

Conclusion from Review of Shield Building Design

The shoulder area was considered an architectural (cosmetic) feature of the shield building and no credit for the shoulders was taken in the design of the structure. The shoulder horizontal reinforcing steel was based upon standard practices and tied back only at the sides of the shoulder area.

As described above, the shield building was analyzed for the appropriately conservative design basis loadings and load combinations described in the USAR. The structure was designed for these loads in accordance with the ACI 307-69 and 318-63 concrete codes and the allowable stress limits of the Davis-Besse USAR. This provided a robustly designed structure.

The robustness of this structure is further demonstrated by the functionality Calculations C-CSS-099.20-054 and C-CSS-099.20-056. These two calculations were performed to evaluate the affect of the identified laminar cracks. The two analyses evaluated the structure on the basis that a large percent of the steel reinforcement was considered to be ineffective. The analyses concluded that even with the "ineffective reinforcement," the shield building remains adequate to perform its design basis functions.

In conclusion, the original Davis-Besse shield building was appropriately designed for the identified loadings and code requirements. The effects on the identified laminar cracking have been evaluated and the shield building remains adequate to perform its safety functions.

However, there was no application of a sealant on the exterior concrete surface of the shield building to protect against moisture migrating into the structure.

3.3.6 Construction

The actual DBNPS site construction was accomplished through twelve major construction contracts and a number of other smaller contracts. The shield building wall was built by Fegles Power Service under construction contract CC-18 and Purchase Order 1221. Bechtel Power Corporation was retained to provide construction management services. Specification C-38 governed the construction of the shield building along with several of the C-100 series of drawings. The concrete forming, placing, finishing, and curing were in accordance with Specification C-26. The reinforcing steel furnishing, detailing, fabricating and delivering were in accordance with Specification C-29. Nicholson Concrete and Supply Company operated the central concrete mix plant in accordance with Specification C-25, and on-site materials testing services was performed by the Pittsburg Testing Laboratory in accordance with Specification C-27. The A. Bentley & Sons Company performed general station structural work including construction of the shield building dome and closure of shield building blockouts that were open after the initial construction for access, and mechanical or electrical penetrations.

The Fegles Power Service proposed construction of the shield building wall using the slip-form method of concrete construction based upon experience from three other similar containment structures. The proposal was based upon revision 0 of the C-100 series of drawings for a shield building wall 139 feet inside diameter, 256 feet 6-½ inches high, and 2 feet 6 inches thick that was estimated to contain 10,881 cubic yards of concrete, and 1715 tons of reinforcing steel.

The shield building specification [Reference 10.1.23] required Type II cement for use in slip-form construction of the shield building below grade, and Type I cement above grade. The Type I cement had earlier strength gain which facilitates the speed of the slip-form construction. Specification C-38 also required shield building construction tolerances for thickness, roundness, plumb, and azimuth / elevation of embedded items and blockouts. Work not included in the shield building construction contract was installation of waterproofing membrane on the exterior of the shield building wall below-grade, or sealant of the shield building wall.

The historical central concrete mix plant specification (C-25) required C-2 class concrete for foundations and walls over 12 inches thick. The C-2 class concrete required a 1-1/2 inch maximum aggregate size, 5 inch slump working limit at the point of placement for slip-formed concrete, a compressive strength of 4000 pounds per square inch at 28 days, and an air content of 3 to 6 percent by volume. The concrete mixes for the shield building slip-form construction were developed by Fegles Power Service and approved by Bechtel Power Corporation. Concrete mix C-2-SF-2 was used below grade for the shield building wall and concrete mix C-2-SF-4 was used above grade.

Slip-form construction enables continuous structures that are based upon the quick-setting properties of concrete, but require a balance between the quick-setting capacity and workability. The concrete form is surrounded by a platform used for workers pouring the concrete, placing reinforcing steel into the form, and staging of materials. The concrete form and working platform are periodically raised together by means of hydraulic jacks. The slip-form rises at a rate which permits the concrete to harden by the time it emerges from the bottom of the form. Concrete used in slip-form construction needs to be workable enough to be placed into a form and packed, yet quick-setting enough to emerge from the form with sufficient strength to permit the form to slip upward and also support additional concrete poured above.

The slip-form method was used in the early 20th century for construction of concrete silos and grain elevators. Other applications for the slip-form construction method include concrete chimneys and stacks, storage tanks, communication towers, bridge foundations, buildings such as the Canadian National (CN) Tower, and containment buildings at nuclear power plants. Slip-form construction continues to be used for construction of the concrete containment walls with the Canadian deuterium uranium (CANDU) nuclear plants.

The major advantages of the slip-form method are a shortened construction schedule, and the ability to complete the project with a single concrete pour with no joints. The slip-form method for construction of CANDU concrete containment walls takes 13 to 20 days versus 6 to 9 months for conventional construction methods. Disadvantages of the slip-form method of construction are increased planning and coordination of resources, very few contractors with slip-form capability, and mistakes are difficult to correct.

The slip-form construction used for the DBNPS shield building consisted of a form four feet high on the inside and outside of the wall with working decks eight feet wide on the inside and six feet on the outside [Reference 10.1.24]. A cement finisher's platform was suspended at both the inside and outside wall faces approximately 9 feet 6 inches below the working deck. This platform was carried on the bottom of the pre-fabricated deck supports. Eighty pairs of 1-1/8 inch diameter mild steel jacking rods equally spaced around the circumference of the wall were used to lift the slip-form as the shield building was constructed. Lifting of the slip-form was accomplished by means of pneumatic jacks that climb the rods and push against the yokes which are attached to the top and bottom of each segment of the slip-form. The yokes hold the form to its proper width and batter. All of the jacks were connected to one control board and operated at the same time to lift the form.

After the first four feet of reinforcing steel was placed for the shield building wall, the moving form was erected and all concrete apparatus, jacks, and hoisting equipment was inspected and approved by the Bechtel Quality Assurance Engineer prior to commencing the first concrete pour. The first four feet of wall was poured with the form standing still, but was poured in the same manner as when the form was being raised to help train crews and to balance the concrete level all around the circle. The concrete was placed directly into the forms from a specially designed round concrete bucket in approximately 9 inch layers evenly around the form, and then worked with electric vibrators. The bucket was hoisted to the deck by means of an electrically controlled, free-standing tower crane after being loaded from a charging hopper. At the foundation level, the charging hoppers were fed by concrete conveyers loaded from the ready-mix trucks.

Once the first concrete in the four foot slip-form had attained the proper set, the jacking operation was started. Thereafter, the rate of the forms vertical movement was controlled by the Slip-Form Superintendent, based on the setting rate of the concrete, placing of reinforcing bars, placing of inserts and openings. A minimum of 18 to 20 inches of firm concrete was maintained in the lower part of the forms to provide support at all times for fresh concrete.

Roundness of the slip-form was controlled using an 8 inch channel rolled to a pre-determined radius placed on the top cross member of the jacking yoke to help maintain the required circular configuration. A system of 16 adjustable cables was installed between the inside form and a prefabricated steel hub located around the tower crane mast. This system provided horizontal adjustment necessary to maintain the circular configuration of the slip-forms. These cables were attached to the form segments and their length was adjusted by manually operated one-ton chain hoists.

The form configuration was checked by the job engineer by eight direct diametric measurements across the inside of the wall at approximately the same times as the plumb readings are made. The wall plumb was measured at 16 equally spaced stations on the moving forms at the inside face of the wall. The readings were taken at eight-hour intervals during the slip-form operation. If deviations from plumb indicated that corrective measures were necessary, the slip-form and working platform level were adjusted to bring the whole structure back to its original position. These adjustments were made through control of individual jacks by the jacking crew and/or use of a telescoping leg and guide wheel system mounted on the jacking yoke.

A freeze / thaw test of the slip-form concrete mix was completed on January 8, 1971, prior to commencing construction of the shield building. A total of 32 cylinders 6 inches diameter by 12 inches long were cured, subjected up to 14 freeze / thaw cycles between 0 and 40 degrees Fahrenheit, and compression tested at intervals up to 28 days. None of the specimens subjected to the freeze / thaw cycles showed any surface defects such as spalling, and all had compression test results greater than 4000 pounds per square inch.

Construction of the shield building commenced on January 25, 1971, and was interrupted on February 4, 1971, at elevation 583 feet 6 inches. This completed the below grade work which was allowed by an Atomic Energy Commission exemption obtained on September 10, 1970. Construction permit CPPR-80 was then issued on March 24, 1971, and shield building construction re-commenced with the second concrete pour on April 26, 1971. Construction of the shield building wall was completed on May 19, 1971, at elevation 801 feet 6 inches. The shield building milestones and shield building slip-form construction sequence are described in Attachments 4 and 5.

The DBNPS shield building wall was constructed 139 feet inside diameter, 256 feet 6-½ inches high, and 2 feet 6 inches thick containing 11,028.5 cubic yards of concrete. Data readily available from two structures at other nuclear sites built using the slip-form method describes similar construction. The Pickering vacuum building built in 7 days 10-1/2 hours in the late 1960's using slip-form construction was 168 feet diameter, 159 feet high, and 3 feet thick containing 9,310 cubic yards of concrete and 952 tons of reinforcing steel. The St. Lucie Unit 2 reactor containment building built in 16-1/2 days in the late 1970's using slip-form construction was 132 feet diameter, 192 feet high, and 3 feet thick using 10,000 cubic yards of concrete. The target climb rate for the slip-form was 9 inches per hour at DBNPS, 9 inches per hour at Pickering, and 6 inches per hour at Saint Lucie Unit 2.

The DBNPS shield building remained uncovered until August of 1973. On August 9, 1973, a pour for the construction of the Shield Building dome ring girder was completed. A series of pours were conducted for the parapet and the dome top slab was completed on October 2, 1973. Closure of the original construction opening commenced on August 6, 1975, and the final pour was completed on December 1, 1975. Specification C-25 required D-1 class concrete for foundations and walls less than 12 inches thick and / or congested reinforcement steel. The D-1 class concrete required a 3/4 inch maximum aggregate size, 3 inch slump working limit at the point of placement, and a compressive strength of 5000 pounds per square inch. The A. Bentley & Sons Company used D-1-4P (no fly-ash) for the shield building dome and D-1-3 (no fly-ash) for closure of the original construction opening. The shield building concrete compression test results and mixes are described in Attachments 6 and 7.

In August 2002, a temporary access opening for replacement of the original reactor pressure vessel head was created using a hydrodemolition process within the area of the original construction opening [Reference 10.1.25]. The temporary access opening was restored in September 2002, with replacement concrete that had a specified compressive strength of 4000 pounds per square inch at seven days to expedite achieving the original shield building design compressive strength results of 4000 pounds per square inch at twenty-eight days [Reference 10.1.26]. The concrete strength for the approved mix design was maintained by limitations to the water/cement ratio. The concrete slump was controlled using a high-range water-reducing admixture to enable increased workability without an increase in the established water/cement ratio of the mix. This ensured that the quality and strength of the hardened concrete was not adversely affected by a permitted slump range of 2 to 6 inches at point of placement.

Noteworthy deviations during construction of the shield building walls were issues such as concrete with the wrong water to cement ratio, concrete with smaller coarse aggregate size, concrete with the wrong type of cement, exceeding shield building wall tolerance for plumb, installation of reinforcing steel, embeds, or reglets, and omission of blockouts. The shield building construction deviations are described in attachment 8.

Three Quality Assurance audits of the Fegles Power Service jobsite were conducted on 2/18/1971, 5/11/1971, and 11/22/1972. These audits concluded that all drawings and documentation were of the latest issue and bearing the necessary Bechtel approval.

Other recently discovered issues involving concrete cover and/or spacing for reinforcing steel at the shield building temporary access opening are identified in Condition Reports 2011-03232, 2011-04973, and 2012-00071. Condition Report 2011-03232 identifies locations where the reinforcing steel at the shield building temporary access opening did not have the minimum 3 inches of concrete cover specified on the design documents. The concrete cover is required by the American Concrete Institute to protect the reinforcing steel from environmental conditions and to provide sufficient embedment. The shield building concrete cover condition was reviewed and a rework disposition specified to obtain the proper depth. Condition Report 2011-04973 identified additional locations at the shield building temporary access opening where the 3 inch concrete cover over reinforcing steel was not provided, and locations where the as-found spacing of reinforcing steel did not conform to the design documents. This condition for reinforcing steel cover and spacing was reviewed and a Repair disposition was specified to revise the design document requirements. Engineering Change Package 10-0458 was revised to provide the modified installation requirements and document the technical basis for the changes to the shield building temporary access opening. Condition Report 2012-00071 was initiated to identify that the evaluation of reinforcing steel spacing at the temporary access opening only considered a maximum concrete aggregate size of 1 inch in lieu of the specified 1-1/2 inch maximum aggregate used in the Shield Building slip-form concrete mix design. The conclusion was that the reinforcing steel spacing was acceptable with the specified 1-1/2 inch maximum aggregate used in the Shield Building slip-form concrete mix design.

Conclusion from Review of Shield Building Construction

The deviations during construction were localized and minor in nature such that there was no evidence that construction methods or materials contributed to the laminar concrete cracking in the shield building walls.

3.3.7 Operational Phase

Twenty-one events were identified in the Operational Phase that could have or did occur during the life of the station that could have initiated the laminar cracking in the shield building. These 21 items can be categorized into the three main groups consisting of Construction Activities, Long Term Events, and Acute Events.

Construction Activities

The failure modes associated with the Construction Activities consist of installation of a shield building opening, modification activities that could adversely impact the structure, and penetrations through the shield building. These failure modes were investigated and eliminated since the location of the laminar cracks do not coincide with any of these activities. In addition, settlement was also ruled out since the shield building foundation bears directly on bedrock.

Long Term Events

The failure modes associated with the Long Term Events such as chemical attack, reinforcing steel corrosion, concrete/reinforcing steel creep, physical attack and vibration were investigated and were eliminated based on the results of the testing performed on the concrete cores. These tests show that there is no evidence of chemical attack, active reinforcing steel corrosion, chronic freeze / thaw or cyclic conditions and physical attack.

There is however, supporting evidence that indicates that the south-west quadrant of the shield building, which is typically at a higher temperature due to solar heating, is more prevalent than other locations of the shield building. Therefore, a detailed finite element analysis was performed [Reference 10.1.16, Exhibit 64] to determine the effects of long term thermal stress cycles with seasonal changes.

The possibility of thermal fatigue damage can be assessed by 1) Stress level caused by temperature gradient, 2) Damage, such as cracking, due to thermal strain variation, and 3) Damage, due to elevated temperature. These items are addressed as follows:

1) Stress level caused by temperature gradient

> The finite element thermal stress analysis showed that the maximum radial stress in the structure is about 300 pounds per square inch during a record hot summer day. This is well below the tensile strength of the concrete and well below the fatigue limit of concrete under cyclic loading. Therefore, thermal stress due to cyclic solar loading can be ruled out.

2) Damage, such as cracking, due to thermal strain variation

> The surface strain of concrete under solar heating is a combination of thermal expansion and drying shrinkage. Thermal expansion creates compressive stress in the surface of concrete. The stress level is well below the compressive strength of the concrete [Reference 10.1.16, Exhibit 56, and 64]. The drying shrinkage creates tensile stress of concrete, which could generate cracking in concrete. Petrographic examination results [Reference 10.1.16, Exhibit 26] showed that the depth of shrinkage cracking is not more than one inch after 40 years of exposure to the environment. This depth is relatively small compared to the depth of concrete cover (3 inches). More importantly, the internal surfaces of many concrete samples were examined by microscopes [Reference 10.1.16, Exhibit 68], and there is no significant amount of micro-cracking in the concrete. This indicated that the small surface shrinkage cracks

did not coalescence to form discrete large cracks. Therefore, thermal strain and shrinkage cracking can be ruled out.

3) Damage, due to elevated temperature.

Concrete properties can change under elevated temperatures where phase changes take place (vaporization of water, decomposition of calcium hydroxide, etc). In the case of solar heating and cooling, the temperature level and the rate of temperature variation are not sufficient to generate significant phase transformation and spalling damage in concrete. Therefore, elevated temperature due to solar heating can be ruled out.

The conclusion of all of these analyses is that the radial stresses due to seasonal changes, thermal gradients, gravity, and wind loads are not high enough to cause laminar cracking in the structure. In addition, based on the analysis performed and the concrete core test results, there is no accumulative aging affects or cyclic events that would indicate on going degradation issues. Therefore, all long term operational phase issues can be ruled out.

<u>Acute Events</u>

Several of the acute events failure modes were eliminated based on the event either not occurring (earthquake), or further evaluation/inspection showed that these were not of a concern (lightning and electrical potential), or the crack locations did not match the activity (hydrodemolition).

Extreme weather related acute events were also considered. Emergency declarations and Licensee Events Reports were reviewed to identify the range of environmental conditions experienced by the Shield Building that could be potentially relevant to the concrete laminar cracking. The site has experienced several of these events, specifically, blizzard like conditions and tornados.

The top three blizzards in recent recorded Ohio history in terms of temperature, wind and duration, occurring near the site were determined to be the 1977, 1978, and the 1994 blizzards. There three blizzard were researched and it was determined that the 1978 blizzard was the most significant of the three. The 1978 blizzard had wide spread rain two days prior to the event, wind speeds up to 105 mph, low temperature of -5°F, snowfall of 12 inches, and duration of 3 days. The 1977 blizzard was the second worst to have occurred at the site.

<u>Blizzard of 1978</u>

The blizzard of 1978 was preceded by warmer than normal weather and precipitation. At 4:30 PM on Wednesday, January 25, 1978, the National Weather Service issued heavy snow warnings, which was subsequently upgraded to a blizzard warning at 9 PM, including the area surrounding DBNPS. A moisture laden low pressure system moving northward from the Gulf of Mexico and a polar low pressure system descending from the Arctic were converging over Ohio and would subsequently pass over the plant.

Early Thursday morning, with temperatures slightly above freezing, and rain / fog, the temperature dropped to near zero within 2 hours at the onset of the Arctic air and remained near 10 degrees Fahrenheit throughout the day. Winds increased to greater than 50 miles per hour with a wind chill of -50 degrees Fahrenheit. The wind gusts of 82 miles per hour at Cleveland Hopkins airport were the strongest ever measured, and an ore carrier J. Burton Ayers on Lake Erie, measured sustained winds of 86 and gusts of 111 miles per hour. At DBNPS, the plant was at 75 percent power in start-up testing when the switchyard breakers for one transmission line opened at 0516 AM on Thursday, January 26, 1978, followed an hour later by a loss of all meteorological instrumentation during the blizzard.

The barometric pressure of 28.28 inches mercury at Cleveland Hopkins airport during the blizzard was the lowest in the 48 contiguous states in the 20^{th} century except for during a hurricane. In southern Ontario a barometric pressure of 28.05 inches mercury was measured, with record low barometric pressures also measured at Toledo, Columbus, Akron, and Detroit. Snow accumulated between 5 and 10 inches over the two day duration of the blizzard with snow drifts of 15 to 20 feet. The combination of duration, wind speed, visibility, and temperature rank the storm on January 26 & 27, 1978 as a severe blizzard. The blizzard of 1978 was examined further as a specific case in the modeling and analysis of the shield building. [Reference 10.1.16, Exhibit 61]

Blizzard of 1977

The blizzard of 1977 was also evaluated to determine its affects on the shield building. However, the blizzard of 1977 was not as severe of an acute event in terms of the key parameters consisting of moisture preceding the event, temperature, wind speed, and duration. The stresses produced from this analysis are significantly less then those produced from the 1978 blizzard. The results show that the radial stresses do not exceed the tensile capacity of the concrete and therefore most likely could not have contributed to the observed crack. [Reference 10.1.16, Exhibit 61]

Tornado of 1998

On Wednesday June 24, 1998, a storm cell was tracking southeast from Michigan and then eastward along Lake Erie, until suddenly shifting southeast, making landfall directly adjacent to the DBNPS site. The DBNPS site was near the center of the storm cell, where cloud elevation and wind-speed were the greatest. The rapidly upward moving air feeding the center of the storm spawned several funnel clouds. At approximately 2044 hours, a tornado touched down onsite in the vicinity of the cooling tower with damage to the switchyard that resulted in a complete loss of offsite power. The turbine building roof sustained a large hole estimated to be 8 feet by 20 feet, along with several turbine roof vents ripped off. The strong winds also resulted in a loss of 9 out of 12 meteorological instruments, plus loss of the fiber-optic and copper communication lines.

The storm cell experienced at the DBNPS site on June 24, 1998, was categorized by the National Weather Service as an F2 tornado, exhibiting winds between 113 and 157 miles per hour. The force of this storm was well within the wind and tornado load design basis for the shield building (300 mile per hour wind and 3 pounds per square inch differential pressure). Licensee Event Report 1998-006 documents the tornado damage to the switchyard causing loss of offsite power.

An analysis was performed for the maximum wind speed from the blizzard of 1978, [Reference 10.1.16, Exhibit 62] using a wind speed of 105 miles per hour. This analysis concluded that the wind velocity of 105 miles per hour resulted in stresses of less than 1 pound per square inch around areas where laminar cracking was observed. Scaling the peak wind velocity up to 157 miles per hour for a F2 tornado would not create stresses significant enough to create the laminar cracking observed. The tornado of 1998 was examined further as a specific case in the modeling and analysis of the shield building [Reference 10.1.16, Exhibit 63].

Conclusions from Review of Operational Phase

Twenty-one different failure modes were reviewed that were associated with events that did or could have occurred during the operational phase of the plant (post design and construction). Testing and research of the design documents ruled out the majority of these failure modes. Extensive computer analyses were performed to develop an understanding of the affects the various load combinations would have on the Shield Building stresses.

All scenarios analyzed produced stresses below the tensile strength of the concrete and as such they could be discounted. However, only one extreme environmental event, the blizzard of 1978 which lasted three days, had the individual characteristics that could produce significant stresses. The analysis for the 1978 blizzard concluded that moisture driven into the outer layers of the shield building followed by near zero temperatures could cause freezing of the water resulting in high radial stresses at the shoulder areas.

The blizzard of 1977, which had less moisture, lower wind speed, and lasted only one day, was also investigated and analyzed. The stresses associated with this condition were less than the 1978 blizzard and could not have caused the laminar cracking.

The tornado of 1998 was investigated and it was concluded that the associated wind was clearly below the threshold to create the laminar cracking observed.

3.3.8 Shield Building Modeling and Analysis

Performance Improvement International (PII) analyzed the shield building for the loading conditions that could not be refuted in the Failure Modes analysis, such as Appendixes V, VI, and VII of Reference 10.1.16. As noted in the PII report, several potential load cases were refuted including; seismic (Failure Mode 3.1), snow/ice (Failure Mode 3.13), dead weight of dome (Failure Mode 1.7). Performance Improvement International used concrete stress and fracture analysis modeling techniques originally developed as part of the Crystal River Unit 3 containment concrete delamination cracking root cause investigation. The modeling and analysis was updated to reflect the design characteristics of the DBNPS shield building. The material properties and failure criteria used in the analysis and modeling were based upon the results of the DBNPS shield building concrete laboratory tests and examinations. The values used for the material properties are presented in Attachment 4 of this report.

The following exhibits from the PII report document the analyses performed to evaluate various loading conditions for the potential to develop stresses that could cause the identified laminar cracks in the shield building. In addition to the PII analyses described below, PII performed other analyses that provide input to these analyses or that are not pertinent to the root cause. The five relevant PII analyses are summarized below:

Reference 10.1.16, Exhibit 51 – Freezing Failure and Rebar Spacing Sensitivity Study

Reference 10.1.16, Exhibit 56 – Structural and Thermal Analysis Investigation

Reference 10.1.16, Exhibit 61 – Stress State during 1978 and 1977 Blizzards

Reference 10.1.16, Exhibit 62 - Stress Analysis Due to 105 MPH Wind Load

Reference 10.1.16, Exhibit 73 – Laminar Cracking Due to 1978 Blizzard

PII developed a detailed three dimensional Finite Element Analysis (FEA) model using the Abaqus software for the structural analysis of the Shield Building. The model included the shield building shoulders so that an accurate representation and analysis of this shield building feature of interest could be completed. This model was used to evaluate various loadings on the structure described in the PII exhibits below.

PII also developed a three dimensional finite element model using the Fluent software. The model included the major structures adjacent to the shield building to allow for the accurate assessment of their affects on this structure. This model was developed for computational fluid dynamics analyses and it was used to evaluate various wind and thermal conditions acting on the shield building. This model was used in several PII analysis exhibits.

The NASTRAN analysis software was used to develop a three dimensional finite element model of the shield building. This finite element model was used to evaluate transient thermal temperatures for the various environmental conditions.

Reference 10.1.16, Exhibit 51 - Freezing Failure and Rebar Spacing Sensitivity Study

A FEA model was developed to evaluate the potential for laminar cracks to propagate within the top 20 feet of the shield building cylindrical wall and at each of the main steam line penetration blockouts. These areas of the shield building were designed with the horizontal hoop reinforcing steel spaced at approximately 6 inches (center to center). The actual distance is further reduced in locations of the lap splices for the horizontal hoop reinforcement. This spacing of the reinforcement results in approximately 3 inches of concrete between the reinforcing steel. The analysis evaluated the potential for the concrete between the horizontal reinforcing steel to crack during a condition of saturated concrete and below freezing temperatures. The analysis was based upon a 0.6 and 1.0 percent void fraction under the horizontal reinforcing steel and a 7 percent expansion of the void due to ice freezing. These values are conservative based on testing of the shield building core bores. Testing measured an actual void fraction of approximately 6 percent which is consistent with the air entrainment requirements for the shield building concrete and the 9 percent expansion of water when it freezes. With a given motivating force, such as elements under horizontal reinforcing steel treated as freezing ice, all of the models of the shield building exterior with 6-inch spacing showed the development of some laminar cracks. The analysis for the 1% void fraction and 7% water expansion model indicates that cracking can occur over a significant area of the shield building cylindrical wall.

This same analysis evaluated reinforcing placed at 12 inch spacing (both horizontal and vertical reinforcing steel) which is typical for most of the remaining exterior surface of this structure. The 12 inch spacing analysis determined that laminar cracks would not propagate with this amount of concrete between the reinforcing steel. Therefore, the tighter spacing of the outer face of structural reinforcing steel such as in the top 20 feet of the shield building and adjacent to openings or blockouts can facilitate propagation of laminar cracking as evident at the main steam line penetration blockouts. This analysis also supports the results of the shield building physical investigation cylindrical shell wall where cracking was not found in the areas with the larger reinforcement spacing.

Reference 10.1.16, Exhibit 56 - Structural and Thermal Analysis Investigation

An analysis was also performed to evaluate the seasonal temperature and wind effects associated with the summer solstice, autumn equinox, winter solstice, and vernal equinox. The analysis evaluated the two solstice cases and the two equinox cases in order to study the affects of solar radiation on this structure. The analysis identified two bounding cases: the summer solstice and winter solstice with an extreme combination of temperature and wind conditions. The prevailing wind direction occurs from the southwest for most of the year, particularly the winter and spring seasons. The temperatures were based upon the atmospheric heat sink surrounding the shield building based upon meteorological data.

The case for the summer solstice (time 7:30 PM, no wind, & daytime temperature 104 degrees Fahrenheit) results in a radial tensile stress of less than 100 pounds per square inch starting at the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder. The case for the winter solstice (time 5:00 AM, 105 mile per hour wind, & temperature -24 degrees Fahrenheit) results in a tensile stress of approximately 190 pounds per square inch starting at the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder. These forces are not sufficient to cause cracking, as the tensile failure stress of the shield building concrete is 600 pounds per square inch, as detailed in Attachment 4 to this report.

This exhibit also documents a finite element analysis that was performed to investigate the potential propagation of the existing laminar cracks in the shield building cylindrical shell area. This analysis considered the summer solstice load case and a postulated 30 foot by 30 foot area laminar crack within the top 20 feet of the shield building wall. This analysis evaluated the southwest portion of the shield building where the laminar cracking was found to be most prevalent. The analysis concluded that for the southwest side of the structure that there was only a marginal increase in the magnitude of radial stress, less than 100 psi. Since the shield building concrete has a tensile failure stress of 600 psi (Attachment 4 of this report), there is insufficient radial stress to propagate cracking in the summer and winter bounding cases.

Reference 10.1.16, Exhibit 61 - Stress State during 1978 and 1977 Blizzards

The blizzards of 1978 and 1977 presented a unique combination of environmental conditions acting on the shield building. In 1978, the actual blizzard was preceded by several days of rain. The shield building was evaluated for the potential to introduce moisture into the shield building shell as described Reference 10.1.16, Exhibit 72.

The analysis describes the use of computational fluid dynamics analyses to calculate the surface temperature of the shield building during the blizzard. The calculated temperatures were used as an input to the Abaqus model for the structural evaluation of the building.

The Abaqus analysis evaluated the structure for the affects of freezing of the entrapped moisture. The analysis model used a subsection of the structure that spanned between the centers of adjacent cylindrical wall panels. This model included two shoulders and the flute area between the shoulders. The shield building model considered the extreme combination of temperature and wind conditions of these blizzards. Additionally, the analysis included the subsequent freezing of the moisture laden concrete using the coefficient of thermal expansion.

The case for the blizzard of 1978 results in a radial stress of approximately 550 pounds per square inch behind the thickest section of the architectural flute shoulder. The case for the blizzard of 1978 also results in a hoop stress of approximately 1200 pounds per square inch adjacent to the outer face of structural reinforcing steel behind the architectural flute shoulder, and a vertical stress of approximately 920 pounds per square inch in the same region.

This analysis concluded that the very high stresses are developed in all three directions. This indicates the damage was likely to have occurred. The analysis shows that the locations of high radial stress from the blizzard of 1978 coincide with the observed laminar crack locations under the thick sections of the architectural shoulders and not in the thinner sections the shield building wall.

In order to evaluate the size of the postulated laminar crack a separate analysis was performed as documented in Reference 10.1.16, Exhibit 73.

The 1977 blizzard was also evaluated in this exhibit of the PII report. A similar analysis was performed for the moisture, wind, and temperature conditions for this blizzard. The analysis for this less severe blizzard resulted in stresses that were insufficient to cause the laminar cracking observed.

Reference 10.1.16, Exhibit 62 – Stress Analysis Due to 105 MPH Wind Load

An analysis was performed for the maximum wind speed during the 1978 blizzard of 105 MPH. Reference 10.1.16, Exhibits 67 and 62, describe the computational fluid dynamic and finite stress analysis used to evaluate the 105 mile per hour (MPH) wind force of the shield building. The results of these analyses determined that the calculated radial stress was less than 1 psi for the 105 MPH wind. The 1998 tornado had a wind speed of approximately 157 MPH, Ref. Failure Mode 3.3 of the PII report. By comparison, the tornado wind could not have generated sufficient stress to cause the laminar cracking based on the radial stress values from the 105 MPH wind analysis. Therefore, neither the 105 MPH wind nor the 1998 tornado caused the identified laminar cracks.

Reference 10.1.16, Exhibit 73 – Laminar Cracking Due to 1978 Blizzard

The analysis contained in this exhibit expands on the blizzard analysis presented in Exhibit 61. This analysis was performed to determine if laminar cracks would actually develop in the high stress areas predicted developed in Reference 10.1.16, Exhibit 61.

A detailed finite element submodel was created to analyze the probability for laminar cracks to develop under the previously discussed blizzard conditions. The submodel used the calculated temperature distribution in the shield building discussed in Reference 10.1.16, Exhibit 61. The material properties used in this analysis were derived in a sensitivity study that determined that strength parameters of 600 psi for the concrete tensile strength and fracture toughness of 0.18 in-lb/in^2. These parameter values are well within the expected ranges for these parameters.

The analysis also describes the coefficient of thermal (CTE) expansion of high moisture concrete used in this evaluation. The CTE is described in Reference 10.1.16, Exhibit 57 and it is used as an input for this finite element analysis.

The finite element submodel spans an approximate width of 23 degrees in the circumferential direction and it includes one flute, one shoulder, and one-half of the adjacent cylindrical shell panel. The blizzard conditions were applied to this model and predictions on the development of laminar cracks was determined.

This analysis concluded that laminar cracks developed mostly at the outer reinforcing steel mat under the thick shoulder regions and not in the thinner sections of the flute and shell. This cracking pattern was determined to have been caused during the 1978 blizzard. The 1977 blizzard determined that damage was significantly less likely to have occurred.

Conclusion from Shield Building Modeling and Analysis

None of the analyses that evaluated the dead, live, seismic, wind, tornado, or historical average or extreme combination of temperature and wind conditions developed sufficient radial stress to cause laminar cracking of the shield building concrete. Therefore, the forces involved with the laminar cracking were beyond those anticipated with the design practices used for the shield building.

The prevailing wind direction occurs from the southwest for most of the year, particularly during the winter and spring seasons, and was the path of the blizzard of 1978.

The postulated acute freezing of moisture adjacent to the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder in conjunction with the blizzard of 1978 was the only scenario capable of producing large stresses. These stress levels would indicate that damage was likely in the shoulder areas where the principle laminar cracking has been identified, Reference 10.1.16, Exhibit 61.

Reference 10.1.16, Exhibit 73 was performed to determine if laminar cracks would actually develop in the high stress areas identified in Reference 10.1.16, Exhibit 61. This analysis documents a detailed finite element submodel of the shield building that was evaluated for the 1978 and 1977 blizzard conditions. This analysis concluded that laminar cracks formed in the shoulder regions during the 1978 blizzard. The analysis concluded that cracking was considerably less likely to have occurred during the 1977 blizzard.

Reference 10.1.16, Exhibit 51 of the PII report describes the analysis of the outer face of structural reinforcing steel for the potential of crack propagation. This analysis determined that a 6 inch or less (center to center) reinforcement spacing would facilitate laminar crack propagation. This analysis is applicable to the laminar cracks found within areas such as within the top 20 feet of the building wall and the areas adjacent to the main steam line penetration blockouts.

Reference 10.1.16, Exhibit 56 also documents an analysis of the shield building cylindrical wall for a postulated 30 foot by 30 foot area laminar cracking within the top 20 feet of the shield building wall. This analysis evaluated a reinforcing steel spacing of 12 inches (center to center) to determine if cracks could propagate. The analysis concluded that there is insufficient radial stress to overcome the tensile strength of the concrete and that no propagation would occur under conditions such as summer and winter extreme temperatures. Based on this analysis, there is no expectation that the existing laminar cracks will propagate.

In summary, the analyses discussed above concluded that the laminar cracks formed in the shield building shoulders during the blizzard of 1978 and that these cracks propagated into the top 20 feet of the shell wall and main steam line penetration blockouts due to the greater density of horizontal hoop reinforcing steel.

3.3.9 Failure Modes Analysis

A fault tree of potential failure modes for the shield building concrete laminar cracking was developed collegially among root cause team members from FENOC, VATIC Associates, MPR, and Performance Improvement International. A list of 45 possible failure modes that could potentially contribute to the laminar cracking, either individually or in concert were identified based upon characteristics of the shield building laminar cracking and other operating experiences with concrete issues.

All 45 failure modes were grouped in three major categories consisting of Design, Construction & Fabrication, and Operational. Each of these failure modes was evaluated during the root cause investigation. In general, each failure mode was either refuted or supported by laboratory tests and examinations or by state of the art analysis. Some failure modes were refuted by deductive reasoning based on existing evidence to either support or refute their mode of failure. Potential failure modes were not eliminated unless there was positive refuting evidence against a given failure mode. Attachment 11 identifies the Fault Tree. Attachment 12 details the supporting and refuting evidence for each failure mode.

Completion of the Equipment Apparent Cause Evaluation form (NOP-ER-1001-01) did not identify any additional failure modes.

<u>Group 1 Failure Modes (Design)</u>

A review of the initial design documents revealed the shield building was conservatively designed, considered all of the required loads and followed the code requirements. The Shield Building design has a large margin when compared to the allowable loads. Except for the reinforcing steel detailing associated with the shoulder area (FM 1.3, and 1.12), good design practices were used. As an example, the specified reinforcing steel to reinforcing steel lap splice (FM 1.4) is greater than specified per the ACI Code. FM 1.5 although acceptable per the code, the area of high density reinforcing steel coincided with observed laminar cracking and could not be ruled out and requires further investigation.

<u>Group 2 Failure Modes (Construction & Fabrication)</u>

A review of the construction drawing, specifications, and test records were evaluated with respect to the work preformed. Reviews of the historical testing records for concrete compressive strength determined that the average compressive strength from 92 cylinder sets exceeded both the 7-day, and 28-day design requirements (FM 2.1). In addition to compressive strength, a review of initial construction records and sample tests were completed to evaluate aggregate strength and placement, cement type, air content (69 tests reviewed, and petrographic analysis), and durability. These evaluations determined there were no indications of reactive or weak inclusions in the aggregate, with relatively well distribution of aggregate in the concrete matrix. In conjunction with no evidence of micro-cracking, freeze / thaw damage (air content evaluation) and confirmatory strength testing it can be concluded that the materials contained no precursors to laminar cracking. (FM 2.1)

Mixing, conveying, placing, finishing, and curing of concrete for the shield building was governed under project specifications. The placement records were reviewed and determined that the placement practices were within acceptable limits. These reviews did not identify any segregation, temperature variations, concrete cover variations, or curing practices that would contribute to the laminar cracking condition. From this review it can be determined that failure modes such as Concrete Mix (FM 2.1), Concrete Placement (FM 2.2) Drying Shrinkage (FM 2.4), and Concrete Construction (FM 2.5) were not the cause of the laminar cracking.

Concrete core taken from the shield building in support of this root cause were also tested to obtain in-situ concrete physical and chemical properties. The test results from these concrete cores also confirm that the concrete is sound and many of the failure modes in this section can be eliminated. Concrete core testing also confirmed that the exterior surface of the shield building did not have a sealant to keep moisture out and examination of the cores revealed evidence of secondary deposits which typically suggest long term exposure to moisture migrating through the concrete (FM 2.7).

Consistent with the finding in Group 1, FM 3.11 Small Rebar Spacing could not be ruled out and requires further investigation.

Group 3 Failure Modes (Operational)

Review of events since the construction of the shield building was performed. The review encompassed items such as Earthquakes (FM 3.1), Tornados (FM 3.3), and Chemical Attack (FM 3.9).

During the review of operational records it was determined that the site had no environmental loading such as seismic activity (FM 3.1), Lightning (FM 3.2), and vibrations (FM 3.14) that would result in laminar cracking.

In order to confirm that wind loading was not the initiating factor of the laminar cracking; the structure was evaluated against the most extreme wind event on record at Davis-Besse. This condition was a category F2 tornado that passed in close proximity to the shield building in June 1998. A Finite Element Analysis model with the tornado conditions was generated to analyze this specific condition. This analysis concluded that wind loading did not generate stresses of the magnitude to cause laminar cracking (FM 3.3).

Since initial construction, the shield building has been subjected to environmental conditions and weathering. The impact of this was analyzed in support of the root cause by conducting examinations of the in-situ concrete structure, and testing of extracted samples. Specific testing on 17 exterior face samples determined that the average exterior carbonation depth was approximately 9 mm. This value represents very low levels considering the life of the structure, and did not compromise the protective concrete cover on the reinforcement which has a nominal thickness of 3 inches (76 mm) (FM 3.9). Twenty-three concrete cores were also tested and inspected for carbonation on the interior crack surface. These inspections found only trace amounts of carbonation and were found acceptable.

Other information gathered under visual in-situ and petrographic examinations indicated that there was no evidence of alkali-silica reactions, freeze / thaw micro-cracks, sulfate attack, leaching and efflorescence, acid degradation, or reinforcement corrosion. This information concludes that Operational Factors such as Chemical Attack (FM 3.9), Corrosion of Rebar (FM 3.10), Physical Attack (FM 3.15) and chronic Freeze / Thaw (FM 3.16) were not the cause of the laminar cracking.

Long term thermal stress cycles (FM 3.7) conditions were investigated [Reference 10.1.16, Exhibits 56, 64, and 68]. The conclusions from these analyses are the radial stresses due to seasonal changes, thermal gradients, gravity, and wind loads are not high enough to cause laminar cracking in the structure. In addition, based on the analysis performed and the concrete core test results, there is no accumulative aging affects or cyclic events that would indicate on going degradation issues. Therefore, all long term operational phase issues can be ruled out.

Freezing near reinforcing steel in a Blizzard (FM 3.6) were also investigated and analyzed. This extreme environmental event associated with the 1978 blizzard could produce high stresses that could initiate laminar cracking in the locations observed.

Conclusion from Failure Modes Analysis

Finite Element Analysis of the shield building wall identified that there is a stress concentration located near the outer layer of reinforcing steel between the flute shoulder area and the shield building shell directly behind the thick section of the architectural flute shoulder.

A review of the design drawings identified that the architectural shoulder horizontal reinforcing steel is connected only at the sides of the shoulders with an approximate 10 foot span. There are no intermediate stirrups or radial reinforcing steel in this region to resist any radial stress in this area should high radial stress occur. (Failure modes 1.3 and 1.12)

Finite Element Analysis of high density reinforcing steel spacing (#6 bar at 6" c-c) indicate that a laminar crack could propagate in this area for a specified motive force, while laminar cracking does not propagate when the reinforcing steel spacing is the normal 12" c-c spacing for the identical motive force. (Failure modes 1.5 and 2.11)

A review of the design drawings and installation specifications revealed that an exterior sealant on the shield building exterior surface was not required. In addition, petrographic examinations of the concrete cores identified that the exterior surface did not have a sealant layer. In addition, laboratory tests showed secondary deposits virtually in all air voids indicating the presences of long term exposure to moisture migrating through the concrete. (Failure mode 2.7)

Finite Element Analysis identified that the acute freezing of moisture adjacent to the outer mat of reinforcing steel directly behind the shoulder areas could produce radial stress greater than the concrete tensile strength when subjected to the environmental conditions associated with the blizzard of 1978. Finite Element Analysis was also done for the 1977 blizzard and the stresses associated with the conditions experienced in this blizzard could not produce the high stress needed to exceed the concrete tensile strength. Therefore, the 1978 blizzard was the only scenario capable to producing radial stress to enable the laminar crack initiation. (Failure mode 3.6)

3.3.10 Hardware Disposition

The initial condition assessment determined that the shield building was functional, but non-conforming with the concrete laminar cracking. The initial condition assessment concluded that no compensatory actions or operating restrictions were required due to the shield building concrete laminar cracking. Engineering analysis demonstrated that the shield building remained structurally adequate for the controlling load case(s). However, the shield building with the laminar cracking in its walls remains non-conforming to the current design and licensing bases with regard to design stress analysis methodology, and the tornado allowable stress values.

Direct Cause Corrective Action #2 re-establishes design and licensing basis conformance for the shield building with the observed concrete cracking.

<u>Design stress analysis methodology</u>

USAR Section 3.8.2.2.5 and DCM Section II.H.2.5.1.5 specify the analysis methodologies used for the shield building design. These documents state that the shield building wall was designed using the American Society of Mechanical Engineers (ASME) "Analysis of Spherical Shells" from Section III of the 1968 code.

The initial condition assessment Calculations C-CSS-099.20-054 and 056 used the "ANSYS" computer analysis code to study the affect of the laminar cracks on the function of the shield building. Any calculations used as the design for the shield building with the concrete laminar cracking will require conformance with the design & licensing bases.

<u>Tornado allowable stress values</u>

USAR Section 3.8.2.2.6 and DCM Section II.H.2.5.1.5 define the load combinations and allowable stresses for the shield building design. Study Calculation C-CSS-099.20-056 documents that the calculated stress for the tornado wind and differential pressure load exceeded the allowable stress value in the design and licensing basis, but was within the allowable limit using the alternate differential pressure design load of Regulatory Guide 1.76, Rev. 1.

<u>Disposition</u>

The primary disposition for the majority of the shield building laminar cracking is "Repair." Design Engineering will prepare and track to completion a comprehensive plan to restore the shield building to conformance with its design and licensing bases requirements.

4 Safety Culture Evaluation

The causal factors for the laminar cracking of the shield building concrete wall were primarily design related from about 40 years ago, so the evaluation of safety culture aspects is not relevant to current performance.

5 Latent Organizational Weakness Evaluation

The causal factors for the laminar cracking of the shield building concrete wall were primarily design related from about 40 years ago, so the evaluation of latent organizational weaknesses is not relevant to current performance.

6 Generic Implications

6.1 Plant and Industry Experience

6.1.1 Strategy

The FENOC Corrective Action Program databases, Institute of Nuclear Power Operation (INPO) Plant Events Database, the Nuclear Regulatory Commission (NRC) website, and Electric Power Research Institute (EPRI) website were searched for similar symptoms of containment shield building concrete laminar cracking from at least the last five years using the keywords Containment, Shield Building, Concrete, Crack, and Hydrodemolition.

A second search was conducted for plant and industry experience with concrete cracking to gain knowledge regarding similar potential failure modes, causes, corrective actions, and generic problems. Search terms (trend codes) from the direct and root causes included blizzard, moisture, sealant, coating (0550), design interface (B04, F04, CM10), and design specification (3900).

6.1.2 Results

There was no previous DBNPS experience with shield building concrete laminar cracking. In 2002, a similar temporary access opening was created using hydrodemolition for the replacement of the reactor pressure vessel closure head. The 2002 temporary access opening was confined within the blockout used for the original construction opening and was not in an area exposed to similar regions where laminar cracks were found in 2011. There were no symptoms of concrete distress observed on the exterior of the shield building that would indicate laminar cracks were located below the surface.

The Perry Nuclear Power Plant has a similar configuration (without architectural features) of shield building as DBNPS, but there is no experience with similar laminar cracks. The Beaver Valley Nuclear Power Station units have a different containment system consisting of a reinforced concrete cylinder with a steel liner and there is no experience with similar laminar cracks.

A document from the NRC regarding containment liner corrosion operating experience [Reference 10.1.27] lists 16 similar locations with a reinforced concrete shield building and a freestanding steel containment vessel. The 14 pressurized water reactors with a reinforced concrete shield building and a freestanding steel containment vessel are Davis-Besse, Kewaunee, Prairie Island 1 & 2, Saint Lucie 1 & 2, Waterford 3, Catawba 1 & 2, McGuire 1 & 2, Sequoyah 1 & 2, and Watts Bar 1. The 2 boiling water reactors with a reinforced concrete shield building and a freestanding steel containment vessel are Perry 1 and River Bend 1.

None of these plants have reported experiencing similar laminar cracks. Davis-Besse and Saint Lucie have created temporary access openings in the shield building wall to replace major components. Sequoyah and Watts Bar have created temporary access openings in the shield building dome to replace major components. Kewaunee, Prairie Island, Catawba, and McGuire have performed their major component replacements through the equipment hatch. Waterford has a major component replacement scheduled in the future.

The majority of the nuclear power stations that have completed major component replacements through temporary access openings in containment systems are either post-tensioned or reinforced concrete cylinders with a steel liner. The only other similar instance of concrete delamination discovery associated with creating a temporary access opening in the containment structure occurred at Crystal River unit 3. The root cause of the Crystal River containment concrete delamination was the design of the structure in combination with the type of concrete used, and the acts of de-tensioning and opening the containment structure.

A study of the deterioration of concrete water storage tanks in the province of Ontario [Reference 10.1.28] identified damage that ranged from heavy surface spalling and cracking to delamination and eventual failure of some structures. The study concluded the prime factors for the determining the rate of concrete structure deterioration were the number of freeze / thaw cycles, temperature amplitudes and frequencies, concrete permeability, hydrostatic pressure, location, the effect of reinforcing steel, and internal ice formation. Remedial solutions proposed included repair of joints and voids, applying waterproof coatings, insulation, and tank replacement.

An NRC document on the durability of reinforced concrete structures [Reference 10.1.29] identified that water is the single most important factor controlling the degradation process of concrete apart from mechanical deterioration. This document was considered in the failure modes analysis. Also considered in the failure modes analysis were an ACI document on the evaluation of nuclear safety-related structures [Reference 10.1.30], an EPRI document on concrete at nuclear power plants [Reference 10.1.31], and an IAEA document on assessment and management of aging of nuclear power plant components [Reference 10.1.32].

An ACI document on concrete cracking causes and restoration [Reference 10.1.33] identified methods of crack repair that may be applicable to the shield building concrete laminar cracking include epoxy injection and additional reinforcement. Another ACI document describes the use of waterproofing barrier systems for concrete [Reference 10.1.34]. These documents were considered in the failure modes analysis and potential corrective actions.

6.1.3 Conclusions

The laminar cracking of the shield building wall is unique with respect to reinforced concrete, but a much more severe symptom of laminar cracking occurred in a post-tensioned concrete containment structure at Crystal River unit 3 due to a combination of design, materials, and the act of de-tensioning.

Similar concrete laminar cracking has occurred in this geographical area with water tanks due to environmental conditions, concrete permeability, and the effect on reinforcing steel. Industry resources identify water as the single most important factor controlling the degradation process of concrete apart from mechanical deterioration. Some solutions proposed included applying waterproof coatings and insulation. Past occurrences of similar conditions with concrete laminar cracking in water tanks, water as a controlling factor in concrete degradation, and applying waterproof coatings as solutions suggest that there may be a broader issue with moisture penetration. The extent of condition review addresses the broader issue with moisture penetration other than the shield building exterior.

There were no similar issues with design specifications, coatings, or design interfaces to indicate a generic problem. There have been no similar previously identified events with concrete laminar cracking at DBNPS or within FENOC to judge the effectiveness of prior corrective actions.

There have been no similar previously identified events with concrete laminar cracking with conventionally reinforced concrete shield buildings to judge the effectiveness of operating experience reviews.

Since the failure modes for the laminar cracking of the shield building concrete wall were primarily design related from about 40 years ago under a quality assurance program outside the control of FENOC, then there is no basis to judge the effectiveness of training, self-assessment, or oversight.

6.2 Extent of Condition

6.2.1 Strategy

The shield building concrete laminar cracking was reviewed for the extent of condition relative to other applicable programs / processes, equipment / systems, organizations, environments, and individuals.

6.2.2 Results

The shield building was the only structure on site designed by Bechtel as a reinforced concrete right cylinder. The shield building was the only nuclear safety-related structure on site constructed using the slip-form process by Fegles-Power Service. The shield building wall laminar cracking was primarily located at the outer face of structural reinforcing steel under the architectural flute shoulder regions due to the concentration of radial stresses. The shield building wall laminar cracking was also across the top 20 feet and adjacent to the main steam line penetration blockouts due to the density of reinforcing steel facilitating propagation. The shield building wall laminar cracking is predominantly oriented to the southwest due to the prevailing direction of severe storms, including the blizzard of 1978.

The shield building was the only above-grade nuclear safety-related structure on-site designed by Bechtel during original construction that did not have a white cement Thoroseal finish for sealing of exterior concrete surfaces [Reference 10.1.35]. A waterproofing membrane was installed below-grade on the shield building exterior. The shield building dome lacks factors found in the architectural flute shoulders like the discontinuity stress concentration factor and high density reinforcing steel necessary for crack initiation and propagation. Therefore, only the remainder of the accessible, above-grade, exterior wall of the shield building should be examined similar to those areas previously examined.

The failure modes for the laminar cracking of the shield building concrete wall were primarily design related from about 40 years ago under a quality assurance program outside the control of FENOC. Therefore, the condition does not currently exist in other applicable programs / processes, equipment / systems, organizations, environments, and individuals.

6.2.3 Conclusions

The extent of condition was adequately bounded by the initial condition assessment [Reference 10.1.19] based upon empirical data available at that time.

Extent of Condition Corrective Action #1: Additional Examination of the Shield Building Exterior Wall.

There is no broader issue with moisture penetration other than the shield building exterior since other above-grade nuclear safety-related structures were sprayed with a white cement finish for sealing the exterior concrete surfaces.

6.3 Extent of Cause

6.3.1 Strategy

The knowledge gained from the industry experience review regarding the causes of concrete cracking was used to develop the potential failure modes for the shield building laminar cracking.

The shield building concrete laminar cracking was reviewed for the extent of cause relative to other applicable programs / processes, equipment / systems, organizations, environments, and individuals.

6.3.2 Results

The shield building was the only above-grade nuclear safety-related structure on site designed by Bechtel during original construction that was not sprayed with a white cement Thoroseal finish for sealing of exterior concrete surfaces. The failure modes for the laminar cracking of the shield building concrete wall were primarily design related from about 40 years ago under a quality assurance program outside the control of FENOC. Therefore, the extent of cause was not reviewed in other programs / processes, equipment / systems, organizations, environments, and individuals.

6.3.3 Conclusions

The accessible exterior concrete surfaces of the shield building should be sealed to prevent moisture penetration like the other nuclear safety-related structures on-site. The exterior of other nuclear safety-elated structures should be examined to ensure the protective coating remains acceptable.

Root Cause Corrective Actions #1 & 2 design and implement a shield building exterior sealant system.

Root Cause Corrective Actions #3 update inspection procedure to include shield building exterior sealant system.

6.4 Data Analysis Conclusions

On October 10, 2011 a concrete crack was observed at the architectural flute shoulder region of a temporary access opening in the shield building wall. The temporary access opening was being cut by supplemental personnel under the direction of FENOC using a hydrodemolition process to allow replacement of the reactor pressure vessel head. A concrete crack in the architectural flute shoulder region of a temporary access opening in the shield building wall was unexpected and needs to be understood to ensure there is no impact with its structural integrity currently, previously, or within the future viable service life of the plant. Previous inspections of the shield building exterior surface did not identify symptoms that would signify the presence of the concrete laminar cracking.

An acoustic sounding of the shield building exterior wall was performed using the Impulse Response testing method to locate areas with concrete laminar cracking. Confirmation of the Impulse Response testing results was achieved by visual inspection of 70 core bores, including crack characterization. The initial condition assessment determined that the shield building concrete wall contained tight-width laminar cracking near the outer face of structural reinforcing steel. The majority of the shield building laminar cracking occurred in the concrete at the outer face of structural reinforcing steel located behind the architectural flute shoulder region. Areas of the laminar cracking occurred beyond the architectural flute shoulder region as evident across the top 20 feet of the shield building and in localized areas adjacent to one side of each main steam line penetration blockout. The southwestern exposure of the shield building wall was observed with the most extensive concrete cracking.

The initial condition assessment determined that the shield building was functional, but non-conforming with the presence of the concrete laminar cracking. Engineering analysis demonstrated that the shield building remained structurally adequate for the case of the controlling loads. However, the shield building with the laminar cracking in its walls remains non-conforming to the current design and licensing bases with regard to design stress analysis methodology, and the alternate tornado design criteria. **Direct Cause Corrective Action #2** re-establishes design and licensing basis conformance for the shield building with the observed concrete cracking.

Examination of 36 shield building concrete cores was performed to define possible failure modes for the laminar cracking, or quantify material properties of the concrete to support computer modeling and analysis. Two laboratories performed Petrographic examination of 4 concrete cores in order to determine the concrete condition, possible reasons for the damage, and prediction of whether deterioration may continue. Four other laboratories examined the shield building concrete cores for material properties and possible failure modes.

The external laboratory examination of the shield building concrete core samples determined that the concrete was in good condition, consistent with the mix design, and no unacceptable or degraded material properties. There was no evidence of typical concrete time-dependent aging failure modes. The integrity of the concrete narrowed the failure mechanism to those related to design or environmental issues versus construction. The examination found the outer surface of the cores was not water-repellant, and the air voids were lined with secondary deposits of ettringite and calcium hydroxide which suggests long-term exposure to moisture migrating through the concrete.

Computer modeling of shield building loading under environmental conditions with extreme combinations of temperature and wind were insufficient to result in laminar cracking of the concrete. Therefore, the forces involved with the laminar cracking were beyond those anticipated with good design practices. The postulated acute freezing and expansion of moisture in the shield building concrete was the only probable scenario capable of radial stresses large enough to enable the laminar crack initiation. The blizzard of 1978 was the only identified event during the life of the shield building that integrated the moisture content, wind speed, temperature, and duration necessary for development of radial stresses large enough to enable the concrete laminar crack initiation.

The most likely reason for the shield building laminar cracking was a lack of exterior protective sealant that allowed moisture to migrate into the concrete, freeze, and expand. The other nuclear safety-related structures on-site have a protective sealant as a barrier against moisture migration into the concrete.

The extent of condition was adequately bounded by the initial condition assessment based upon empirical data available at that time. The remainder of the accessible shield building exterior walls should be examined using Impulse Response testing with confirmatory core bores to clearly define the extent of condition. The exterior concrete surfaces of the shield building should be sealed as a barrier against moisture migration like the other nuclear safety-related structures on-site.

Causal Factors

Finite element analysis identified that the acute freezing of moisture adjacent to the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder in conjunction with the blizzard of 1978 was the only scenario capable of radial stresses to enable the laminar crack initiation (failure mode 3.6).

Petrographic examination of the concrete cores and review of the design records identified that the shield building exterior lacks a sealant and experienced long-term exposure to moisture migrating into the concrete (failure mode 2.7).

Finite element analysis of the outer face of structural reinforcing steel identified that a 6-inch or less spacing of reinforcement facilitated laminar crack propagation (failure modes 1.5 & 2.11).

Review of design records identified that although the architectural flute shoulder reinforcing steel included connections to the structural reinforcing steel at the ends approximately 10 feet apart, there were no middle stirrups or radial reinforcing steel to tie the reinforcement elements together for crack mitigation (failure modes 1.3 & 1.12).

TapRoot®

The path through the TapRoot® Root Cause Tree® for the shield building concrete laminar cracking is Equipment Difficulty / Design / Design Specifications / Specifications Need Improvement.

The reason for the shield building laminar cracking was that the design specification for construction (C-038) of the shield building did not specify application of an exterior sealant from moisture. The lack of exterior sealant enabled moisture preceding the blizzard of 1978 to migrate into the concrete, freeze, and expand.

A contributor to the shield building laminar cracking was an inherent stress concentration at the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder. The stress concentration behind the thickest section of the architectural flute shoulder enabled the radial stress from the freezing moisture to exceed the tensile strength of the concrete and initiate a crack. A horizontal (hoop) stress and vertical stress that adjoined the outer face of structural reinforcing steel underneath the architectural flute shoulder region enabled the laminar crack created by freezing moisture to propagate.

A second contributor to the shield building laminar cracking was the design did not include radial reinforcing steel ties or stirrups at an intermediate spacing between each end of the architectural flute reinforcing steel connection with the structural reinforcing steel. The design not including intermediate radial reinforcing steel ties or stirrups enabled the laminar crack created by freezing moisture to propagate.

A third contributor to the shield building laminar cracking was a density of structural reinforcing steel less than or equal to six inch spacing. Once the crack originated in the shoulder region, it continued to propagate into adjacent areas where the higher density of reinforcing steel was present such as at the top 20 feet of the shield building. The greater density of structural reinforcing steel enabled the laminar crack created by freezing to propagate into this area. The main steam line penetration blockout laminar cracks propagated due to the freezing in adjacent shoulders located three feet above the auxiliary building roof and the higher density reinforcing steel. This cracking continued until obtaining equilibrium and stopping at these blockouts.

7 Root and Contributing Causes

The **Direct Cause** for the shield building concrete laminar cracking is the integrated affect of moisture content, wind speed, temperature, and duration from the blizzard of 1978. [Cause code T22]

The environmental conditions from the blizzard of 1978 enabled radial stresses from moisture to freeze and expand, creating radial stresses which then exceeded the tensile strength of the concrete and initiated the cracking in the shield building exterior wall. The blizzard of 1978 was the only event identified during the life of the shield building that integrated the moisture content, wind speed, temperature, and duration necessary for development of radial stresses large enough to enable the concrete laminar crack initiation.

In equipment failure analysis, a root cause is typically the main factor that leads to the damage of an engineered system. A root cause must possess the following characteristics to be considered valid: 1) it can be eliminated to prevent recurrence, and 2) it is within the control of management. This investigation has identified several factors that when combined resulted in the laminar cracking observed. Specifically shoulder configuration, reinforcing steel details, environmental conditions, and no exterior sealant from moisture. Each of these factors can be eliminated as not under the control of management except one, and that is the root cause.

The **Root Cause** for the shield building concrete laminar cracking was due to the design specification for construction of the shield building (C-038) that did not specify application of an exterior sealant from moisture. [Cause code DA1D]

The design specification for the shield building identified work that was not included in the slip-form construction such as backfill and installation of waterproofing membrane below grade, but did not specify application of an exterior sealant from moisture. The other nuclear safety-related structures on-site have a protective sealant as a barrier against moisture migration into the concrete.

The examination found the outer surface of the cores was not water-repellant, and the air voids were lined with secondary deposits of ettringite and calcium hydroxide which suggests long-term exposure to moisture migrating through the concrete.

Computer modeling of the shield building under environmental conditions with extreme combinations of temperature and wind were insufficient to result in laminar cracking of the concrete. The acute freezing and expansion of moisture in the shield building concrete was the only scenario capable of generating radial stresses large enough to enable the laminar crack initiation.

A subsequent regulatory reference on the durability of reinforced concrete [Reference 10.1.29] from February, 2007 identified water as the single most important factor controlling the degradation process of concrete apart from mechanical deterioration.

Contributing Cause #1 for the shield building laminar cracking was an inherent stress concentration at the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder. [Cause code DA1D]

The stress concentration behind the thickest section of the architectural flute shoulder enabled the radial stress from the freezing moisture to exceed the tensile strength of the concrete and initiate a crack. Other horizontal (hoop) and vertical stresses that adjoined the outer face of structural reinforcing steel underneath the architectural flute shoulder region enabled the laminar crack, once created by freezing moisture, to propagate along the outer face of structural reinforcing steel.

Contributing Cause #2 for the shield building laminar cracking was the design did not include radial reinforcing steel ties or stirrups at intermediate spacing between each end of the architectural flute shoulder reinforcing steel connection with the structural reinforcing steel. [Cause code DA1D]

The design not including radial reinforcing steel ties or stirrups at intermediate spacing enabled the laminar crack created by freezing moisture to propagate to the end connections.

Contributing Cause #3 for the shield building laminar cracking was a density of structural reinforcing steel less than or equal to six inch spacing. [Cause code DA1D]

Once the crack originated in the shoulder region, it continued to propagate into adjacent areas where the higher density of reinforcing steel was present such as at the top 20 feet of the shield building, and the main steam line penetration blockouts. The greater density of structural reinforcing steel enabled the laminar crack created by freezing moisture to propagate into these areas.

8 Corrective Action Plan

Problem Statement On October 10, 2011 a concrete crack was observed at the architectural flute region of a temporary access opening in the shield building wall.

Extent of Condition Corrective Action #1: Additional Examination of the Shield Building Exterior Wall.

Site Projects shall arrange access to the exterior face of the shield building wall. This access will be used to support further investigation of the structure. Engineering will specify the areas of access required and the necessary work scope, such as additional Impulse Response and core bores). Using an Impulse Response (IR) vendor and method approved by Design Engineering identify potential cracked or un-cracked areas of the Shield Building as directed by Design Engineering.

Provide the necessary ground and/or suspended man-lifts required to access the shield building wall exterior surface.

Perform confirmatory core bores as directed by Design Engineering

Facilitate the examination of the core bores by Design Engineering.

Repair/Rework core drill holes as described in the ECP for the core bore

Extent of Condition Corrective Action #2: Issue Engineering Change Package for Additional Shield Building Core Bores.

Design Engineering to issue an Engineering Change Package (ECP) to allow for additional core bores, as required, in the exterior surface of the Shield Building.

This ECP shall identify the size, location, reinforcing steel cutting allowances (if any), and maximum number of core bores for this issue. This ECP shall also revise the applicable design documents (drawings, calculations, etc.) to track the core bores, crack depths, crack widths etc.

The **Direct Cause** for the shield building concrete laminar cracking is the integrated affect of moisture content, wind speed, temperature, and duration from the blizzard of 1978.

Direct Cause Corrective Action #1: Testing Program to Investigate the Steel Reinforcement Capacity Adjacent to Structural Discontinuities.

Design Engineering will administer a testing program to be performed at a selected test facility. The test procedure will be developed and performed by the selected facility to determine the effect on the steel reinforcement/ splices adjacent to structural discontinuities (i.e. laminar cracks). This test program and the deliverable test report will be reviewed and approved by Design Engineering.

Direct Cause Corrective Action #2: Engineering Plan to Re-Establish Design & Licensing Basis for Shield Building.

Design Engineering to develop a comprehensive plan for re-establishing shield building conformance to the DBNPS design and licensing bases. Upon the completion of the corrective action for inspection of the shield building, the extent of laminar cracking will be established. Upon the completion of the corrective action for the Testing Program the capacity of the reinforcing steel adjacent to the laminar cracking will be known. The steps for re-establishing shield building design and licensing bases conformance will be finalized and additional corrective actions will be initiated as required.

Direct Cause Corrective Action #3: Issue a Site Specific Procedure for the Long-Term Monitoring of the Shield Building Laminar Cracking.

Design Engineering will develop procedural requirements for the long-term monitoring of the Shield Building laminar cracking condition. The procedural requirements will include the following:

1. The periodic monitoring of the shield building will begin with an annual inspection cycle starting in 2012. The schedule is outlined below:

   2012 inspection shall be completed by 9/1/2012

   2013 inspection shall be completed by 9/1/2013

   If the inspection results remain unchanged after the first two inspection cycles (defined as no discernable change in crack width or the confirmation that no cracks have developed in previously un-cracked core bores), the inspection cycle will change to two year cycles per the schedule below:

   2015 inspection shall be completed by 9/1/2015

   2017 inspection shall be completed by 9/1/2017

   2019 inspection shall be completed by 9/1/2019

   The periodic monitoring will repeated every two years for a minimum of three cycles. If after three monitoring cycles, the results of the core bore and crack examinations remain unchanged, the monitoring schedule may be changed to a five year cycle.

   2024 inspection shall be completed by 9/1/2024

   2029 inspection shall be completed by 9/1/2029

   2034 inspection shall be completed by 9/1/2034

   If any adverse changes are identified (as defined in the acceptance criteria) during these examinations, a Condition Report shall be initiated to evaluate any required change to the inspection schedule.

2. A minimum of 6 core bores of each type (un-cracked & cracked) will be inspected during each inspection cycle. The approximate distribution of the core bore inspections is as follows: 3 in shoulder regions, 1 in the steam line penetration areas, and 2 at the top of the building outside the shoulders.

3. The examination of the core bores will be performed by visual inspection and the use of a boroscope and optical crack comparator. Any identified cracks shall be measured using an optical crack comparator and the boroscope.

4. The acceptance criteria to be used for the examination of the core bores that did not contain a crack indication initially (as defined on Drawing C-111A) shall be that no new crack indication is identified. If a new crack is identified, a Condition Report shall be initiated and the crack shall be measured as described above. The condition report will determine any additional inspections of required changes to the monitoring.

   The previously identified core bores containing cracks shall be re-examined to determine the current width of the crack. The as-measured crack width shall be compared to the initial crack width measurement as recorded on Drawing C-111A. If it is it determined that the crack width has increased (a discernable change in width within the accuracy of the measurement technique), a Condition Report shall be initiated to evaluate.

5. Chloride ion testing and carbonation testing will be carried out on a minimum of 2 core samples collected for examination. This testing will be performed during alternating inspection cycles. The testing will be performed as detailed in the requirements. The procedural requirements will establish the acceptance criteria for these tests.

The **Root Cause** for the shield building concrete laminar cracking was due to the design specification for construction of the shield building (C-038) that did not specify application of an exterior sealant from moisture.

Root Cause - Preventive Action #1: Issue Engineering Change Package for a Shield Building Exterior Sealant System.

Design Engineering to issue an Engineering Change Package (ECP) to specify the required details and requirements for application of a sealant to the exterior of the shield building. The selected system will preclude moisture migration into the reinforced concrete. As part of this ECP, establish a preventive maintenance frequency once the specific product is selected for the shield building exterior sealant system.

Root Cause - Preventive Action #2: Implement Engineering Change Package for a Shield Building Exterior Sealant System.

Site Projects shall provide for the implementation of the shield building sealant system specified in the Engineering Change Package. This sealant system shall be applied to exterior of the shield building, as specified in the Engineering Change Package.

Root Cause - Corrective Action #3: Update Inspection Procedure to Include Shield Building Exterior Sealant System.

Design Engineering shall update the Maintenance Rule Structures evaluation procedure (EN-DP-01511) for inspection of the shield building exterior sealant system.

Contributing Cause #1 for the shield building laminar cracking was an inherent stress concentration at the outer face of structural reinforcing steel behind the thickest section of the architectural flute shoulder.

Contributing Cause #1 – Corrective Action: None required.

Basis for no action required: The stress concentration behind the thickest section of the architectural flute shoulder enabled the radial stress from the freezing moisture to exceed the tensile strength of the concrete and initiate a crack. Other horizontal (hoop) and vertical stresses that adjoined the outer face of structural reinforcing steel underneath the architectural flute shoulder region enabled the laminar crack created by freezing moisture to propagate along the outer face of structural reinforcing steel. The root cause preventive actions for an exterior sealant system established a barrier against moisture migrating into the concrete. The acute freezing and expansion of moisture in the shield building concrete was the only scenario capable of radial stresses large enough to enable the laminar crack initiation. Computer modeling of shield building loads under environmental conditions with extreme combinations of temperature and wind were insufficient to result in laminar cracking of the concrete without moisture migration and subsequent freezing. Therefore, the root cause preventive actions nullify the impact of contributing cause #1.

Contributing Cause #2 for the shield building laminar cracking was the design did not include radial reinforcing steel ties or stirrups at intermediate spacing between each end of the architectural flute reinforcing steel connection with the structural reinforcing steel.

Contributing Cause #2 – Corrective Action: None required.

Basis for no action required: The design not including radial reinforcing steel ties or stirrups at intermediate spacing enabled the laminar crack created by freezing moisture to propagate to the end connections. The root cause preventive actions for an exterior sealant system establish a barrier against moisture migrating into the concrete. The acute freezing and expansion of moisture in the shield building concrete was the only scenario capable of radial stresses large enough to enable the laminar crack initiation. Computer modeling of shield building loads under environmental conditions with extreme combinations of temperature and wind were insufficient to result in laminar cracking of the concrete without moisture migration and subsequent freezing. Therefore, the root cause preventive actions nullify the impact of contributing cause #2.

Contributing Cause #3 for the shield building laminar cracking was a density of structural reinforcing steel less than or equal to six inches at the top 20 feet of the shield building, and at openings or penetrations.

Contributing Cause #3 – Corrective Action: None required.

Basis for no action required: The greater density of structural reinforcing steel enabled the laminar crack created by freezing moisture to propagate beyond the end of the architectural flute reinforcing steel connection with the structural reinforcing steel such as that evident at the top 20 feet of the shield building and adjacent to the main steam line penetration blockouts. The root cause preventive actions for an exterior sealant system establish a barrier against moisture migrating into the concrete. The acute freezing and expansion of moisture in the shield building concrete was the only scenario capable of radial stresses large enough to enable the laminar crack initiation. Computer modeling of shield building loads under environmental conditions with extreme combinations of temperature and wind were insufficient to result in laminar cracking of the concrete without moisture migration and subsequent freezing. Therefore, the root cause preventive actions nullify the impact of contributing cause #3.

Confirmatory Action Letter Commitment – Corrective Action #1: Root Cause Report Submittal.

FENOC (Design Engineering) will provide the results of the shield building concrete laminar cracking root cause evaluation and corrective actions to the NRC, including any long-term monitoring requirements, by February 28, 2012.

Confirmatory Action Letter Commitment – Corrective Action #2: Examine Four Un-Cracked Core Bores Following Restart.

FENOC (DBNPS Design Engineering) will examine four un-cracked core bores directly adjacent to locations that have been confirmed to be cracked with a boroscope to verify cracking has not migrated to these core bores located in solid (un-cracked) concrete, within 90 days following plant restart (Mode 2) from the October 2011 mid-cycle outage.

a. adjacent to a flute shoulder [S9-666.0-12]
b. in a flute area [F4-1-666.0-3]
c. adjacent to main steam line penetration 39 [S7-652.0-6.5]
d. adjacent to main steam line penetration 40 [S9-650.0-9]

Confirmatory Action Letter Commitment – Corrective Action #3: Main Steam Line Room New Core Bore & Examination Following Restart.

FENOC (DBNPS Site Projects) will perform a core bore in a known crack area within the main steam line room and Design Engineering will examine the crack interface to identify any changes, within 90 days following plant restart (Mode 2) from the October 2011 mid-cycle outage.

Confirmatory Action Letter Commitment – Corrective Action #4: Examine Six Un-Cracked Core Bores in 17RFO.

FENOC (DBNPS Design Engineering) will examine six un-cracked core bores directly adjacent to locations that have been confirmed to be cracked with a boroscope to verify cracking has not migrated to these core bores located in solid (un-cracked) concrete, during the seventeenth refueling outage currently scheduled to commence in 2012.

a. adjacent to a flute shoulder [S9-666.0-12]
b. in a flute area [F4-1-666.0-3]
c. adjacent to main steam line penetration 39 [S7-652.0-6.5]
d. adjacent to main steam line penetration 40 [S9-650.0-9]
e. in a flute area [F5-777.0-4]
f. adjacent to a flute shoulder [S2-783.5-4.0]

Confirmatory Action Letter Commitment – Corrective Action #5: Examine Three Crack Interface Core Bores in 17RFO.

FENOC (DBNPS Design Engineering) will examine the crack interface to identify any changes by examining either existing core bore locations with known cracks or by performing a core bore in a similar area during the seventeenth refueling outage currently scheduled to commence in 2012.

a. adjacent to a flute shoulder [S9-666.0-11]
b. near the top of the shield building [S9-785-22.5]
c. adjacent to main steam line penetration [S9-653.0-9] (core bore following restart)

9 Effectiveness Review Plan

Prerequisites for this effectiveness review:

After at least one operating cycle of implementing the site specific procedure for the long-term monitoring of the shield building laminar cracking and completion of the following corrective actions.

Direct Cause Corrective Action #1: Testing Program to investigate the Steel Reinforcement Capacity adjacent to structural discontinuities (i.e. laminar cracks)

Direct Cause Corrective Action #2: Engineering Plan to Re-Establish Design/Licensing basis for Shield Building

Direct Cause Corrective Action #3: Issue a site specific procedure for the long-term monitoring of the shield building laminar cracking

Root Cause - Preventive Action #1: Issue Engineering Change Package for a shield building exterior sealant system

Root Cause - Preventive Action #2: Implement Engineering Change Package for a shield building exterior sealant system

Root Cause - Corrective Action #3: Update Inspection Procedure to Include Shield Building Exterior Sealant System.

In accordance with NOBP-LP-2011 section 4.7.4:

Complete a Maintenance Rule Structures evaluation inspection of the shield building exterior sealant system per procedure (EN-DP-01511) to ensure the moisture barrier is still effective with no areas of unacceptable degradation.

10 References

10.1 Developmental References

10.1.1 DBNPS Updated Safety Analysis Report - Section 3.8.2, "Containment Structures," revision 28, December 29, 2010.

10.1.2 DBNPS Drawing C-110, "Shield Building Roof Plan, Wall Section & Details," revision 6, June 3, 1976.

10.1.3 DBNPS Engineering Change Package 10-0458, "Install Shield Building Construction Opening."

10.1.4 FirstEnergy Order 200433294, "Shield Building Construction Opening in Support of RVCH Replacement."

10.1.5 American Concrete Institute, "Concrete Removal Using Hydrodemolition," ACI RAP Bulletin 14.

10.1.6 American Hydro, "Hydrodemolition Parameter and Sequence (Davis-Besse)," October 28, 2011.

10.1.7 Bechtel Condition Report 25539-000-GCA-GAMG-00182, "Fractured Concrete Found at Shield Building Construction Opening," October 10, 2011.

10.1.8 FENOC Condition Report 2011-03346, "Fractured Concrete Found at 17M Shield Building Construction Opening," October 10, 2011.

10.1.9 CTL Group, "Proposed Work Plan for Davis-Besse Nuclear Power Station," Revision 1, October 16, 2011.

10.1.10 American Concrete Institute, "Nondestructive Test Methods for Evaluation of Concrete in Structures," ACI 228.2R-98, June 24, 1998.

10.1.11 American Society for Testing and Materials, "Standard Test Method for Low Strain Integrity Testing of Piles," D5882

10.1.12 Performance Improvement International, "Root Cause Assessment - Crystal River Unit 3 Containment Concrete Delamination," August 10, 2010.

10.1.13 DBNPS, "Davis-Besse Shield Building Cracking Investigation and Assessment Report," Revision 1, November 23, 2011.

10.1.14 DBNPS Drawing C-111A, "Shield Building Exterior Developed Elevation" revision 2, December 5, 2011.

10.1.15 CTL Group, "Laboratory Evaluation of Shield Building Concrete Cores A and D," October 27, 2011.

10.1.16 Performance Improvement International, "Root Cause Assessment - Davis-Besse Shield Building Laminar Cracking," revision 0, February 23, 2012.

10.1.17 DBNPS Calculation VC02/B001-005, "Shield Building - Thermal Stress," revision 0, October 17, 1977.

10.1.18 DBNPS Calculation VC02/B001-007, "Shield Building - Cylinder Walls," revision 0, October 17, 1977.

10.1.19 DBNPS, "Davis-Besse Shield Building Investigation and Technical Summary," revision 1, November 21, 2011.

10.1.20 DBNPS Calculation C-CSS-099.20-054, "Evaluation of Shield Building for the Permanent Condition with Outside Vertical Reinforcement Removed at Cracking Areas," revision 3, December 1, 2011.

10.1.21 DBNPS Calculation C-CSS-099.20-055, "II/I Evaluation for Architectural Flute Shoulder," revision 0, October 31, 2011.

10.1.22 DBNPS Calculation C-CSS-099.20-056, "Evaluation of Shield Building Hoop Reinforcement with Observed Cracking," revision 2, December 5, 2011.

10.1.23 DBNPS Specification C-038, "Shield Building," revision 1, October 30, 1970.

10.1.24 Fegles Power Service, "Quality Assurance and Construction Procedures – Reactor Shield Wall Slip-Form Construction Method," 7749-C-38-3-1, September 8, 1970.

10.1.25 DBNPS Engineering Change Package 02-0146, "Containment Structure Access Opening for Reactor Pressure Vessel Head Replacement."

10.1.26 FirstEnergy Order 200008657, "Restore the Containment Shield Building."

10.1.27 Nuclear Regulatory Commission, "Containment Liner Corrosion Operating Experience Summary: Technical Letter Report – Revision 1," August 2, 2011.

10.1.28 Golder Associates and W. M. Slater & Associates Inc., "Deterioration and Repair of Above Ground Concrete Water Tanks in Ontario, Canada – Report to Ontario Ministry of the Environment," September 1987.

10.1.29 Nuclear Regulatory Commission, "Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures – A Review of Pertinent Factors," NUREG/CR-6927, February 2007.

10.1.30 American Concrete Institute, "Evaluation of Existing Nuclear Safety-Related Concrete Structures," ACI 349.3R-02, June 17, 2002.

10.1.31 Electric Power Research Institute, "Program on Technology Innovation: Concrete Civil Infrastructure in United States Commercial Nuclear Power Plants," 1020932, May 2010.

10.1.32 International Atomic Energy Agency, "Assessment and Management of Aging of Major Nuclear Power Plant Components Important to Safety – Concrete Containment Buildings," IAEA-TECDOC-1025, June 1998.

10.1.33 American Concrete Institute, "Causes, Evaluation, and Repair of Cracks in Concrete Structures," ACI 224.1R-07, March 2007.

10.1.34 American Concrete Institute, "A Guide for the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete," ACI 515.1R-79.

10.1.35 DBNPS Design Criteria Manual, Materials and Building Finishes, revision 4, August 18, 2004.

10.2 Other References

Condition Reports

2011-03232, Shield Building Reinforcement Bar Concrete Cover Less Than Drawing Requirement

2011-03996, Extent of Condition for Shield Building Fracture Indications

2011-04190, Surface Cracks Identified on Fluted Areas of the Shield Building

2011-04214, Core Bore Found Additional Crack in Architectural Flute Area

2011-04402, Fractured Concrete Found at 17M Shield Building at Main Steam Line Penetrations

2011-04507, Isolated Crack Indication Identified by Impulse Response Testing

2011-04648, Shield Building Impulse Response Indications Above Elevation 780

2011-04973, As-Found Concrete Cover and Spacing of Reinforcement Steel (Rebar) Do Not Meet Specified Design Requirements at the 17M Shield Building Construction Opening

2011-05475, Concrete Cracking at the Top of the Shield Building Wall

2011-05648, Concrete Cracking in Shoulder 4 / Flute 2 Region of the Shield Building (Azimuth 67.5)

Drawings

C-100, Shield Building Foundation Plan & Details

C-109, Shield Building Roof Plan and Details

C-111, Shield Building Wall Development

C-112, Shield Building Details, Sheet 1

C-113, Shield Building Details, Sheet 2

C-114, Shield Building Dome Framing Plan and Details

C-115, Shield Building Blockout Details

E-401, Shield Building Lighting and Lightning Protection

Licensee Event Reports

1978-017, Loss of Meteorological System

1998-006, Tornado Damage to Switchyard Causing Loss of Offsite Power

Potential Condition Adverse to Quality Reports

95-0395, Shield Building Concrete Cracks and Spalling

Procedures

DB-PF-03009, Containment Vessel and Shield Building Visual Inspection

EN-DP-01511, Design Guidelines for Maintenance Rule Evaluation of Structures

EN-DP-01512, Shield Building Concrete Examination

Purchase Orders

55113470, CTL Group

55113539, Performance Improvement International

Specifications

C-25, Central Concrete Mix Plant

C-26, Forming, Placing, Finishing, and Curing of Concrete

C-27, Material Testing Services

C-29, Furnishing, Detailing, Fabricating, and Delivering of Reinforcing Steel

Technical Specifications

3/4.6.1, Primary Containment

Institute of Nuclear Power Operations

INPO Event Report Level 4 11-4, Lessons Learned from Construction Projects Involving Concrete Placement

Nuclear Regulatory Commission

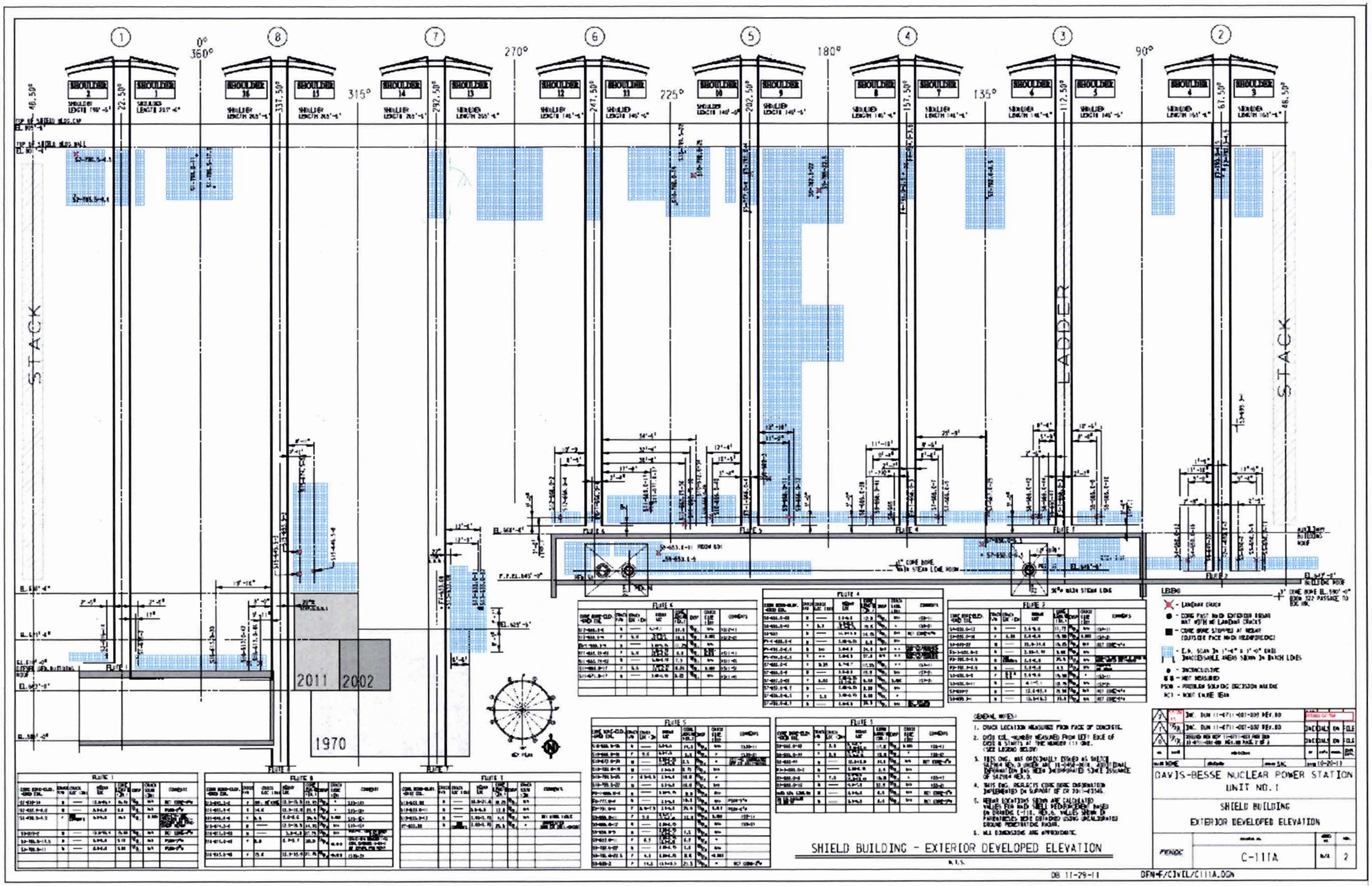
Information Notice 2008-17, Construction Experience with Concrete Placement

Information Notice 2011-20, Concrete Degradation by Alkali-Silica Reaction

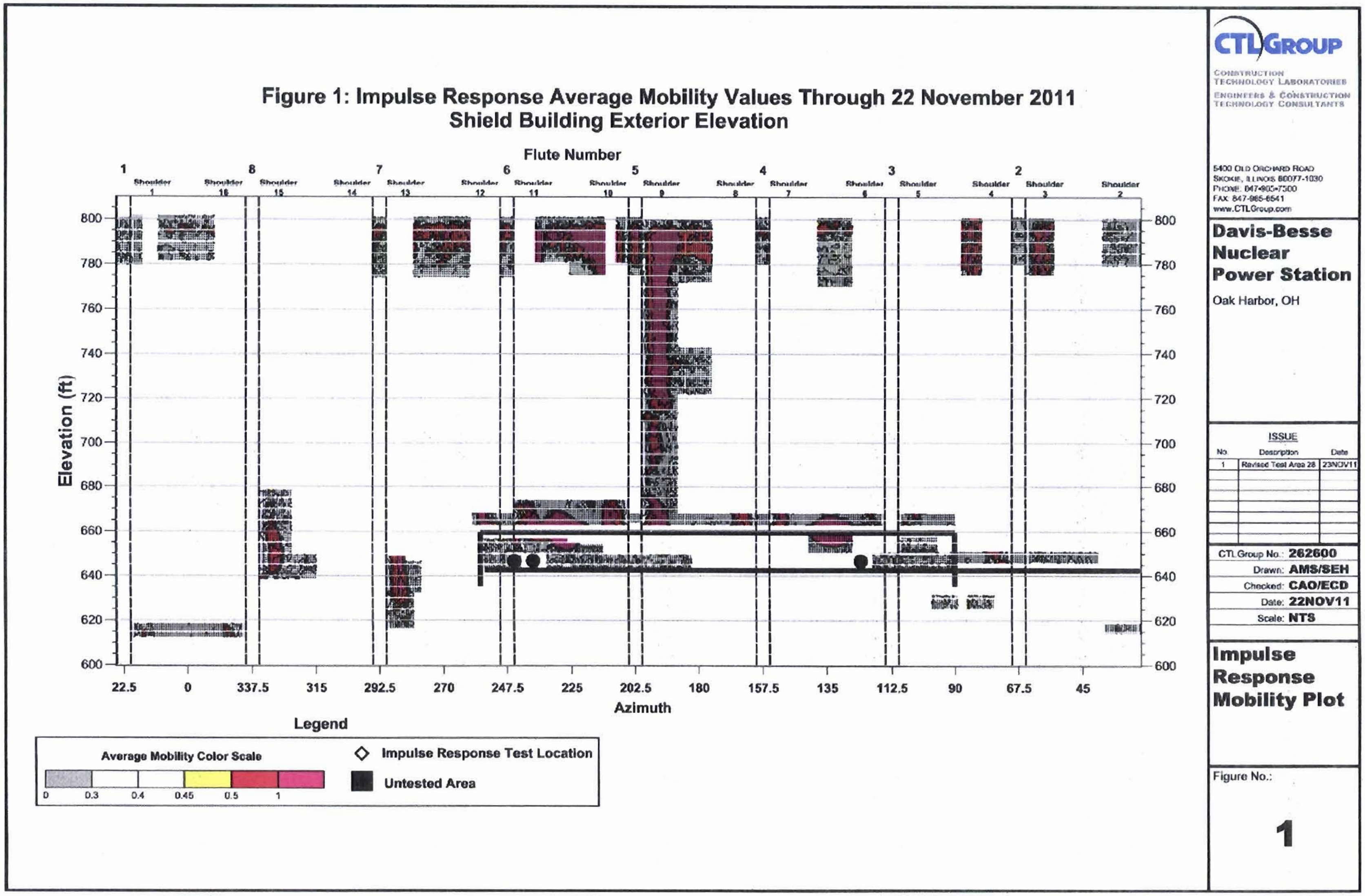
11 Attachments

1. Shield Building Exterior Developed Elevation
2. Shield Building Impulse Response Average Mobility Values
3. Shield Building Core Bore Summary
4. Shield Building Concrete Material Properties
5. Shield Building Surface Visual Inspection History
6. Shield Building Milestones
7. Shield Building Slip-Form Construction Sequence
8. Shield Building Concrete Compression Test Results
9. Shield Building Concrete Mixes
10. Shield Building Construction Deviations
11. Fault Tree
12. Failure Modes Analysis
13. Change Analysis
14. Barrier Analysis
15. Event and Causal Factors Chart
16. Generic Implications Matrix
17. Corrective Action Matrix

Attachment 1, Shield Building Exterior Developed Elevation



Attachment 2, Shield Building Impulse Response Average Mobility Values



Attachment 3, Shield Building Core Bore Summary

Core Bore Number (alias)	Core Diameter (inches)	Core Depth (inches)	Crack Depth (inches)	Crack Width (inches)	Comments
S1-615-2 (S1)	4	15	No crack	Not applicable	Twining Compressive MOE
S1-786.5-17.5	2	9.75	No crack	Not applicable	
S1-798.0-11	2	9	No crack	Not applicable	
S2-616-14 (S2)	4	15.75	No crack	Not applicable	Colorado CTE Creep
S2-783.5-4.0	2	9	No crack	Not applicable	
S2-798.5-4.5	2	10	5	<0.005	PhotoMetrics Carbonation
F2-1-650.0-3	2	9	No crack	Not applicable	
F2-790.0-4.5	3	25.5	No crack	Not applicable	PhotoMetrics Carbonation Void size
F2-792.3-4.5	3	4.5	No crack	Not applicable	WJE Petrographic
S3-650.0-9	2	15.5	8 & 9	Inconclusive	
S3-650.0-11	2	12.75	No crack	Not applicable	
S3-699.3-1 (S3-55)	3	18	No crack	Not applicable	PhotoMetrics Carbonation
S3-650-2 (S3)	4	15.5	No crack	Not applicable	Twining/Colorado Split tensile
S4-650.0-13	2	11.75	No crack	Not applicable	
S4-650.0-16	2	15	6	0.009	
S4-649-22 (S4)	4	15.25	No crack	Not applicable	Colorado Compressive CTE
F3-1-666.0-3 (F3-1)	2	8.5	No crack	Not applicable	PhotoMetrics Carbonation Void size Aggregate size
S5-666.0-8 (S5-1)	2	16	7	Inconclusive	PhotoMetrics Carbonation Void size
S5-666.0-10 (S5-2)	2	12.5	No crack	Not applicable	PhotoMetrics Carbonation Void size
S6-666.0-42	2	17	9	0.005	
S6-666.0-44	2	19	9	Inconclusive	
S6-665-47 (S6)	4	14	No crack	Not applicable	Colorado Relative humidity
F4-1-666.0-3	2	8	No crack	Not applicable	

Core Bore Number (alias)	Core Diameter (inches)	Core Depth (inches)	Crack Depth (inches)	Crack Width (inches)	Comments
F4-791.0-2.5	3	24	No crack	Not applicable	USBR/WJE Specific heat CTE Petrographic
F4-794.0-3.5	3	27.6	Not measured	Not measured	PhotoMetrics Carbonation
S7-666.0-7 (S7-1)	2	17.25	9.25	Not measured	PhotoMetrics Carbonation Void size Aggregate size
S7-666.0-9 (S7-2)	2	15.5	No crack	Not applicable	PhotoMetrics Carbonation Void size
S7-667.0-25 (S7-3)	2	6.5	6.5	0.007	PhotoMetrics Carbonation Void size
S7-782.0-8.5	3	26	No crack	Not applicable	USBR Thermal Diffusivity
S7-652.0-6.5	2	8	No crack	Not applicable	
S7-656.0-6.5	2	8	5	Inconclusive	PhotoMetrics Carbonation Void size Aggregate size
S8-666.0-38	2	12	No crack	Not applicable	
S8-666.0-41	2	15	6	Inconclusive	
S8-665 (S8)	4	14.75	No crack	Not applicable	Colorado Split tensile
F5-1-666.0-4	2	8	No crack	Not applicable	
F5-777.0-4	2	10	No crack	Not applicable	
F5-791.0-4	2	25.5	6.5 - 7.5	0.013	PhotoMetrics Carbonation
S9-666.0-11 (S9-1)	2	13	5	0.005	PhotoMetrics Carbonation Void size
S9-666.0-12 (S9-2)	2	10.5	No crack	Not applicable	PhotoMetrics Carbonation Void size
S9-782.5-22	2	7.5	No crack	Not applicable	
S9-785.0-22.5	2	8	4	<0.010	PhotoMetrics Carbonation Void size Aggregate size
S9-650.0-9	2	7.5	No crack	Not applicable	

Core Bore Number (alias)	Core Diameter (inches)	Core Depth (inches)	Crack Depth (inches)	Crack Width (inches)	Comments
S9-653.0-11	2	6.5	6.3	Inconclusive	PhotoMetrics Carbonation Void size Aggregate size
S9-680-3 (S9)	3	21.5	14.5	Inconclusive	Colorado Compressive Freeze / thaw
S10-666.0-38	2	14	No crack	Not applicable	
S10-666.0-40	2	5	5	Inconclusive	
S10-672.0-34	2	2.5	No crack	Not applicable	
S10-780.0-19	2	8.75	No crack	Not applicable	
S10-790.5-25	2	10	4.5 - 5.5	Inconclusive	
S10-799.5-22	2	10	No crack	Not applicable	
F6-1-666.0-4	2	11.25	No crack	Not applicable	
S11-663.75-30 (S11-1)	2	8	5	0.005 – 0.010	PhotoMetrics Carbonation Void size
S11-663.75-32 (S11-2)	2	7.5	No crack	Not applicable	PhotoMetrics Carbonation Void size
S11-669.0-17	2	8.25	5.5	0.005	
S11-671.0-17	2	8.25	No crack	Not applicable	
S12-666.0-2 (S12-1)	2	21	No crack	Not applicable	PhotoMetrics Carbonation Void size
S12-666.0-4 (S12-2)	2	16.5	5	0.005	PhotoMetrics Carbonation Void size
F7-633.08	2	25.5	No crack	Not applicable	
S13-633.08	2	18.25	No crack	Not applicable	PhotoMetrics Void size
S13-633.0-11	2	13	No crack	Not applicable	
S13-633.0-12	2	4.5	No crack	Not applicable	
S15-645.5-3 ("A")	2	15.75	15.75	Inconclusive	CTL Group Petrographic
S15-653.5-3 ("D")	2	25.5	14	Inconclusive	CTL Group Petrographic
S15-646.5-8	2	25.5	6.5	0.009	
S15-674.5-3 ("G")	2	14.75	No crack	Not applicable	Colorado
S16-613.0-30	2	27.75	No crack	Not applicable	
S16-613.0-42	2	30.00	8	<0.010	

Core Bore Number (alias)	Core Diameter (inches)	Core Depth (inches)	Crack Depth (inches)	Crack Width (inches)	Comments
S16-613.0-46 (S16-3)	2	21.75	15	<0.010	PhotoMetrics Carbonation Void size
MS room	3	8	No crack	Not applicable	Colorado Freeze / thaw
EDG passage	3	8	No crack	Not applicable	Colorado Compressive

NOTE: Measurement of crack width was inconclusive in several bores due to the affect of the drilling equipment disturbing the crack surface in combination with the tight diameter of the hole complicating use of a crack comparator and boroscope.

Attachment 4, Shield Building Concrete Material Properties

Material Properties for Davis-Besse Overall Global Finite Element Model [FEM] Idealizations

Revised 12 Jan 2012

USBR Test Values per e-mail memorandum 11 January 2012

	Thermal Properties					Mechanical Properties			Failure Criteria	
	29-Nov-11	11-Jan-12	11-Jan-12	11-Jan-12		11-Jan-12	9-Dec-11	29-Nov-11	12-Jan-12	12-Jan-12
Temperature	Density, w[a]	Conductivity, K[b]	Diffusivity, α[b]	Specific Heat, c_p	[1] Emissivity, ε	Thermal Expansion, ϒ	Young's Modulus, E_c	Poisson's Ratio, ν[a]	Failure Stress, f'_t	Fracture Energy, G_f
(°F)	(lb/in^3)	(Btu/hr-in-°F)	(in^2/hr)	Btu/lb-°F[4]		in/in/°F ($x10^{-6}$)	lbs/in^2 ($x10^6$)		(lbs/in^2)	(lbs/in)
≤ 50	0.0868	0.308	7.632	0.478	0.93	5.20	4.94	0.20	600	0.54
100	"	0.243	6.624	0.428	"	"	"	"	"	"
≥ 150	"	0.185	5.760	0.378	"	"	"	"	"	"
	(a.) 150 lb/ft^3 CR3 Legacy Value per Dr. Xi	(b.) Conversion to inch units	$\alpha = K/(w^*c_p)$			USBR e-mail note dated 11 Jan 12	f'_c = 7,500 psi Ave Telecom 9 Dec 11	Legacy Value per Dr. Xi		

Generic Steel - Rebar & Inner Steel Containment

Eckert & Drake, "Heat & Mass Transfer"

	Thermal Properties					Mechanical Properties		
Temperature	Density, w	Conductivity, K	Diffusivity, α	Specific Heat, c_p	[2] Emissivity, ε	[3] Thermal Expansion	Young's Modulus, E	Poisson's Ratio, ν
(°F)	(lb/in^3)	(Btu/hr-in-°F)	(in^2/hr)	Btu/lb-°F[4]		in/in/°F ($x10^{-6}$)	lbs/in^2 ($x10^6$)	
80	0.282	2.670	86.074	0.110	0.25	6.80	29.00	0.30
			$\alpha = K/(w^*c_p)$		(2.) Assumed LCS	(3.) Legacy CR3		

Table 1. Concrete & Steel

4.) Note: When material density is in Mass Units $c_p = c_p * 386.4\ in/sec^2$

Stefan-Boltzmann Constant for Radiation 1.190E-11 $Btu/hr\text{-}in^2\text{-}{}^\circ R^4$

Notes on Radiation used on Transient Thermal Analysis

For thermal transient heat transfer models radiation between the steel and the concrete is treated as a simple radiation to ambient temperature of 120°F. It was assumed the steel temp was invariant. The view factor was set to 1 and the absorptivity of the concrete was set to 0.6. This avoids iterate on the element by element view factor which speeds up the transient analysis. Also this avoids having to consider the properties of the steel for radiation.

Note: Material property values for submodels representative of specific regions may reflect localized measurements.

Attachment 5, Shield Building Surface Visual Inspection History

09/02/11 A DB-PF-03009 inspection identified minor concrete spalling on the shield building exterior in the area from the emergency diesel generator rooms south to the auxiliary building. Condition Report 2011-01540 describes one small area of distressed concrete (minor flaking/spalling) observed on the shield building exterior that has been identified in previous inspections and Condition Report 07-29203. Comparing the pictures from the previous inspection to the current pictures shows no evidence of growth or change in the appearance of the affected area. Resolution of the deficiency is being tracked by Order 200288911.

10/25/07 A DB-PF-03009 inspection identified minor concrete spalling or flaking on the shield building exterior in the area from the emergency diesel generator rooms south to the auxiliary building. Condition Report 07-29203 describes two small areas of distressed concrete 6 inch by 10 inch wide and 2 inch deep located on the northwest side of the shield building at the adjacent to the temporary access opening from 2002 for the replacement of the reactor pressure vessel head. Resolution of the deficiency is being tracked by Order 200288911.

10/17/05 A Maintenance Rule Structure Evaluation identified minor concrete spalling in concrete repairs on the Shield Building exterior surface similar to that identified in the previous evaluation. Cracks were noted and judged to be acceptable.

11/22/02 A DB-PF-03009 inspection identified concrete spalling in four areas on the shield building exterior above the original construction opening that ranged from 2 to 6 inch diameter. Resolution of the deficiency is being tracked by Order 200011687.

05/08/00 A DB-PF-03009 inspection identified concrete spalling with less than two inch depth on the northwest side of the Shield Building exterior between the west and northwest architectural flutes.

06/11/99 A Maintenance Rule Structure Evaluation identified minor concrete spalling mainly with past repairs located above the original construction opening, and various small hairline cracks and in the shield building exterior concrete.

05/18/96 A DB-PF-03009 inspection determined there was no unacceptable degradation on the shield building exterior concrete surface. A grout repair from a previous problem report (PCAQ 95-0395) was holding up well and all other areas were in good order.

10/23/91 A DB-PF-10309 / DB-PF-03009 inspection determined there was no unacceptable degradation identified on the shield building exterior concrete surface.

09/26/88 A DB-PF-03009 inspection determined there was no apparent change in the appearance of the shield building exterior concrete, and no indication of cracking, chipping or other unacceptable degradation.

Attachment 6, Shield Building Milestones

Date	Description
May 22, 1969	The Toledo Edison Company instructed Bechtel to proceed with a containment system for the station utilizing a free-standing containment vessel surrounded by a reinforced concrete shield building instead of the pre-stressed, post-tensional concrete containment.
Aug 01, 1969	The Toledo Edison Company and Cleveland Electric Illuminating Company filed for the necessary licenses to construct and operate the Davis-Besse Nuclear Power Station (DBNPS).
Aug 25, 1969	The Bechtel Power Corporation establishes the shield building walls as 2 feet 6 inches thick with a 2 feet thick hemispherical dome.
Jun 02, 1970	The Bechtel Power Corporation determines that the shield building will be changed to having a shallow dome versus a hemispherical dome.
Jun 04, 1970	The Toledo Edison Company requested an exemption to 10CFR Part 50.10(b) from the Atomic Energy Commission to permit certain work at the site of DBNPS prior to issuance of a construction permit.
Aug, 07, 1970	The Bechtel Power Corporation issued for detailing and material purchase construction civil drawings (C-110 through C-113) for the shield building wall.
Sep 08, 1970	Fegles-Power Service Incorporated revised the procedure for slip-form construction of the DBNPS shield building wall.
Sep 10, 1970	The Atomic Energy Commission granted an exemption to allow concrete and reinforcing steel placement for construction of the shield building and auxiliary buildings up to grade level (583 feet 6 inch elevation).
Oct 09, 1970	Fegles-Power Service Incorporated submitted revised proposal for construction of the shield building wall.
Oct 13, 1970	Construction Contract CC-18 addendum #3 issued to Fegles-Power Service Incorporated for construction of the shield building wall.
Oct 23, 1970	The Bechtel Power Corporation issued for construction civil drawings (C-110 through C-113) for the shield building wall.
Oct 30, 1970	The Bechtel Power Corporation issued for construction the design specification (C-38) for the shield building wall.
Dec 07, 1970	Fegles-Power Service Incorporated accepted Construction Contract CC-18 for construction of the shield building wall.
Dec 29, 1970	The Bechtel Power Corporation approved Fegles-Power Service Incorporated drawings for construction of the shield building wall.
Jan 25, 1971	Fegles-Power Service Incorporated started concrete pours for construction of the shield building wall at the 545 feet elevation.
Feb 04, 1971	Fegles-Power Service Incorporated curtailed concrete pours for construction of the shield building wall at the 583 feet 6 inch elevation.

Date	Description
Mar 24, 1971	The Atomic Energy Commission issued Construction Permit number CPPR-80 for DBNPS.
Apr 26, 1971	Fegles-Power Service Incorporated resumed concrete pours for construction of the shield building wall.
May 19, 1971	Fegles-Power Service Incorporated ended concrete pours for construction of the shield building wall at the 801 feet 6-1/2 inch elevation.
Jun 17, 1971	Fegles-Power Service Incorporated submitted "as-built" drawings for construction of the shield building wall.
Dec 09, 1971	The Bechtel Power Corporation issued for construction civil drawing (C-109) for the shield building roof plan and details.
Mar 02, 1972	The Bechtel Power Corporation issued for construction civil drawing (C-114) for the shield building dome framing plan and details.
Dec 14, 1972	The Bechtel Power Corporation issued for construction civil drawing (C-115) for the shield building blockout details.
Aug 09, 1973	The A. Bentley and Sons Company performed a concrete pour (P1714Q) for construction of the shield building dome ring girder.
Aug 22, 1973	The A. Bentley and Sons Company performed a concrete pour (P1745Q) for construction of the shield building dome bottom slab.
Aug 29, 1973	The A. Bentley and Sons Company performed the first concrete pour (P1758Q) for construction of the shield Building dome parapet.
Sep 12, 1973	The A. Bentley and Sons Company performed the second concrete pour (P1786Q) for construction of the Shield building dome parapet.
Sep 21, 1973	The A. Bentley and Sons Company performed the third concrete pour (P1809Q) for construction of the shield building dome parapet.
Oct 02, 1973	The A. Bentley and Sons Company performed a concrete pour (P1827Q) for construction of the shield building dome top slab.
Aug 06, 1975	The A. Bentley and Sons Company started the concrete pours (P2666Q) from elevation 579 feet to 594 feet 8 inches for closing the shield building construction opening.
Oct 6, 1975	The A. Bentley and Sons Company performed a second concrete pour (P2746Q) from elevation 594 feet 8 inches to 610 feet 4 inches for closing the shield building construction opening.
Dec 01, 1975	The A. Bentley and Sons Company finished the concrete pours (P2826Q) from elevation 610 feet 4 inches to 625 feet 6 inches for closing the shield building construction opening.
Apr 22, 1977	The Nuclear Regulatory Commission issued Facility Operating License NPF-3, Docket 50-346 for the Davis-Besse Nuclear Power Station.
Apr 27, 1977	Fuel loading completed.
Aug 12, 1977	Initial criticality achieved.

Date	Description
Aug 20, 1977	Zero power physics testing completed.
Sep 02, 1977	15 percent power level achieved.
Nov 14, 1977	40 percent power level achieved.
Dec 21, 1977	75 percent power level achieved.
Jan 26, 1978	A severe blizzard impacted power distribution and transportation in the region and rendered the meteorological monitoring system inoperable at the plant.
Apr 04, 1978	100 percent power level achieved.
Jul 07, 1978	Full commercial operation commenced.
Jun 24, 1998	A tornado touched down onsite damaging the switchyard and resulting in a loss of offsite power.
Aug 12, 2002	The American Hydro Company started hydrodemolition (Order 02-003545-010) of the shield building to create a temporary access opening for replacement of the reactor pressure vessel closure head with one from the cancelled Midland Unit 2.
Aug 17, 2002	The American Hydro Company finished hydrodemolition (Order 02-003545-010) of the shield building.
Sep 24, 2002	The Bechtel Power Corporation performed a concrete pour (Order 200008657) from elevation 601 feet 6 inches to 620 feet for restoring the shield building temporary access opening.
Oct 07, 2011	The American Hydro Company started hydrodemolition (Order 200433294) of the shield building to create a temporary access opening for replacement of the reactor pressure vessel closure head with one constructed from Alloy 690.
Oct 10, 2011	Unexpected concrete crack within the shield building temporary access opening.

Attachment 7, Shield Building Slip-Form Construction Sequence

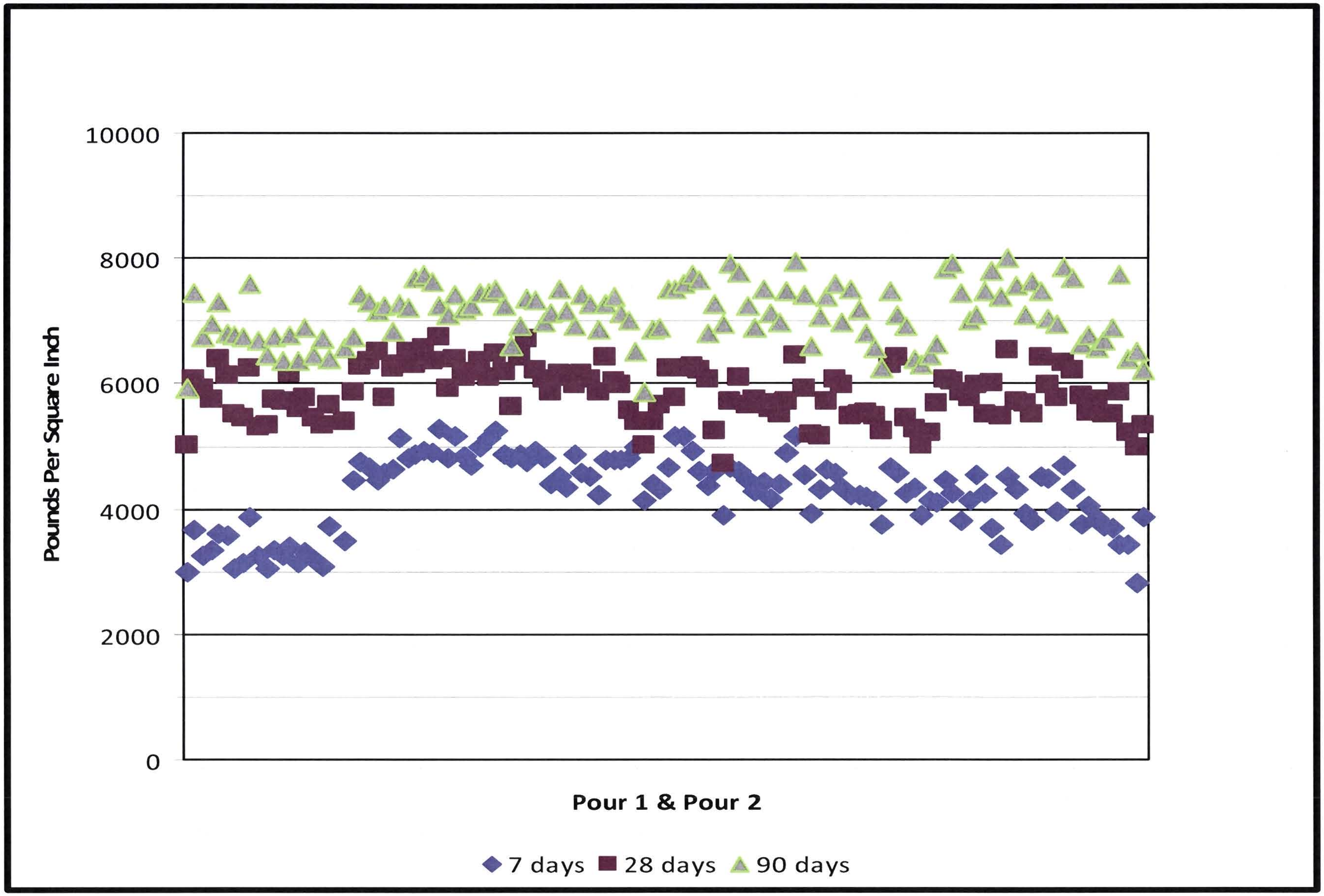
Date	Shift	Deck Height	Footage jacked	Shift Concrete	Total Concrete	Comments
1/25/71	1	4'0"	0	108	108	6 cubic yards concrete batch ticket DB-00219 rejected with 8-1/4 inch slump at 0945 am.
1/25/71	2	5'8"	1'8"	132	240	
1/25/71	3	8'8"	3'0"	108	348	6 cubic yards concrete batch ticket DB-00266 rejected with 7 inch slump at 0130 am.
1/26/71	1	10'8"	2'0"	42	390	Pour stopped at 10 am due to high winds.
2/1/71	1	11'10"	1'2"	120	510	
2/1/71	2	14'10"	3'0"	138	648	
2/1/71	3	17'8"	2'10"	108	756	
2/2/71	1	19'8"	2'0"	108	866	
2/2/71	2	21'9"	2'1"	102	968	6 cubic yards dumped on 2^{nd} shift because of tower crane down time.
2/2/71	3	24'9"	3'0"	114	1082	
2/3/71	1	28'8"	3'11"	162	1244	
2/3/71	2	31'11"	3'3"	132	1376	
2/3/71	3	34'2"	2'3"	90	1466	
2/4/71	1	37'6"	3'4"	132	1598	
2/4/71	2	38'6"	1'0"	66	1664	Pour stopped at 583'6" elevation. Waterstop inserted and key way poured.
Below-grade work for Shield Building construction allowed to 583'6" by a September 10, 1970 Atomic Energy Commission exemption obtained prior to construction permit issuance on March 24, 1971.						
4/26/71	1	43'4"	3'9"	150	1814	**Interim Field Report #1**
4/26/71	2	47'5"	4'1"	144	1958	

Date	Shift	Deck Height	Footage jacked	Shift Concrete	Total Concrete	Comments
4/26/71	3	51'0"	3'7"	126	2084	
4/27/71	1	55'0"	4'0"	138	2222	
4/27/71	2	59'3"	4'3"	132	2354	
4/27/71	3	63'6"	4'3"	150	2504	
4/28/71	1	68'7"	5'1"	168	2672	
4/28/71	2	73'6"	4'11"	156	2828	
4/28/71	3	77'0"	3'6"	144	2972	
4/29/71	1	81'7"	4'7"	168	3140	
4/29/71	2	85'5"	3'10"	156	3296	
4/29/71	3	90'0"	4'7"	204	3500	
4/30/71	1	94'4"	4'9"	210	3710	
4/30/71	2	99'8"	4'11"	198	3908	
4/30/71	3	103'11"	3'5"	156	4064	
5/3/71	1	108'6"	5'5"	216	4280	
5/3/71	2	113'2"	4'8"	192	4472	
5/3/71	3	117'6"	4'4"	192	4464	
5/4/71	1	122'0"	4'6"	198	4862	
5/4/71	2	126'6"	4'6"	204	5066	
5/4/71	3	130'3"	3'9"	180	5246	
5/5/71	1	135'0"	4'9"	228	5474	
5/5/71	2	139'10"	4'10"	222	5696	
5/5/71	3	143'6"	3'8"	180	5876	
5/6/71	1	148'5"	4'11"	216	6092	

Date	Shift	Deck Height	Footage jacked	Shift Concrete	Total Concrete	Comments
5/6/71	2	153'3"	4'10"	228	6314	
5/6/71	3	158'4"	5'1"	228	6542	
5/7/71	1	163'4"	5'0"	234	6776	
5/7/71	2	167'10"	4'6"	204	6980	
5/7/71	3	171'10"	4'0"	204	7184	
5/10/71	1	176'9"	4'11"	216	7400	
5/10/71	2	181'3"	4'6"	198	7598	
5/10/71	3	185'10"	4'7"	204	7802	
5/11/71	1	189'5"	3'7"	180	7982	
5/11/71	2	193'6"	4'1"	180	8162	
5/11/71	3	197'10"	4'4"	210	8372	6 cubic yards of concrete was sent back to the batch plant due to time factor (governed by the spec) due to a break down in the tower crane.
5/12/71	1	202'2"	4'4"	192	8564	
5/12/71	2	205'11"	3'9'	174	8738	
5/12/71	3	210'4"	4'5"	198	8932	
5/13/71	1	214'5"	4'1"	180	9116	
5/13/71	2	218'3"	3'10"	174	9290	**Interim Field Report #3**
5/13/71	3	222'6"	4'3"	192	9482	
5/14/71	1	226'10"	4'4"	195	9677	
5/14/77	2	230'9"	3'11"	180	9857	
5/14/71	3	234'2"	3'5"	181.5	10,038.5	On 2nd shift, truck #82 ticket DB02764 delivered 6 cubic yards of concrete with Type II cement instead of Type I cement.

Date	Shift	Deck Height	Footage jacked	Shift Concrete	Total Concrete	Comments
5/17/71	1	238'8"	4'6"	192	10,230.5	
5/17/71	2	242'6"	3'10"	156	10, 386.5	
5/17/71	3	246'9"	4'3"	192	10,578.5	
5/18/71	1	250'4"	3'7"	168	10,746.5	
5/18/71	2	253'0"	2'8"	108	10,854.5	
5/18/71	3	256' 61/2"	3'6 ½"	108	10,962.5	On 5-18-71 about 9:30 pm the concrete mix was noted as being sticky and not as consistent a mix as it should be. The slump was 3". The problem appeared to be the cement – to try to correct the problem the mix was changed to type II cement at about 11:30 pm 5-18-71. **Nonconformance Report #57**
5/19/71	1	256' 61/2"	0'0"	66	11,028.5	Concrete struck off @ 256'0 ½" & 256'6 ½" water stop and keyway in place and water is being piped to the top of the shield wall for curing the concrete for required time. **Nonconformance Report #359**

Attachment 8, Shield Building Concrete Compression Test Results



Attachment 9, Shield Building Concrete Mixes

	C-2-SF-2	C-2-SF-4	D-1	490A
Specified Strength	4000 pounds per square inch @ 28 days	4000 pounds per square inch @ 28 days	5000 pounds per square inch @ 28 days	4000 pounds per square inch @ 7 days
Cement	Medusa Type II 564 pounds	Type I 588 pounds	Medusa Type II 520 pounds	Type I 490 pounds
Fly ash			Detroit Edison 91 pounds	
Fine aggregate	Woodville Lime manufactured sand 1475 pounds	Woodville Lime manufactured sand 1440 pounds	Woodville Lime manufactured sand 1380 pounds	Roundlake #2 natural sand 1535 pounds
Coarse aggregate	Woodville Lime #67 limestone 930 pounds	Woodville Lime #67 limestone 940 pounds	Woodville Lime #67 limestone 1650 pounds	STONECO #57 limestone 1741 pounds
Coarse aggregate	Woodville Lime #4 limestone 620 pounds	Woodville Lime #4 limestone 620 pounds		
Water	Potable 36.0 gallons	Potable 36.0 gallons	Potable 35.2 gallons	Toledo 28.5 gallons
Admixture	Master Builders Pozzolith 200-N	Master Builders Pozzolith 200-N	Master Builders Pozzolith 200-N	Master Builders Micro Air
Admixture	Master Builders MBVR AEA	Master Builders MBVR AEA	Grace Daravair R	Master Builders Rheobuild 1000
Slump	4 inches	5 inches	4-1/2 inches	5 inches
Air content	5.5 percent	5.7 percent	3.3 percent	4.5 percent

Attachment 10, Shield Building Construction Deviations

Interim Field Report #1

Water cement ratio of mix C-2-SF-4 was exceeded for 48 cubic yards of concrete placed on 4-26-1971 at elevation 583 feet 6 inches in the shield building wall. Minimum temperature was below the specified requirement of 70 degrees Fahrenheit as per the attached concrete cylinder test reports for cylinders 170, 171, 172 and 175. Reference Specifications C-38 & C-25. The attached cylinder strength reports and mix plant inspection report indicate acceptable compression strengths were attained.

Bechtel Engineering has reviewed the Interim Field Report and its attachments relating to an excess of water in concrete mix C-2-SF-4. All concrete breaks are considerable higher than the 4000 pounds per square inch specified. No other harmful effects have been noted in the subject concrete. Bechtel Engineering approves the Use As Is disposition for the structure as it is constructed.

Interim Field Report #3

Fegles Power Services Incorporated placed 6 cubic yards of C-1-3 concrete in pour #2 on 5-13-1971 at deck height 215 feet 6 inches in the shield building wall. Reference Specifications C-38 & C-25. Fly ash was not used in the batch. The batch plant operator apparently did not change the batch plant mix design punch card before producing the aforementioned concrete. The mix design is approved for use in Q-list structures and for 4000 pounds per square inch compression strength requirements. The concrete batch ticket was checked and reveals acceptable quantities of all materials used to produce the concrete in question.

Pittsburg Testing Laboratory report on concrete cylinder numbers 275, 276, 277, and 278 compressive strength indicate that the concrete inadvertently placed in the shield building meets the minimum strength of 4000 pounds per square inch with considerable margin. No other concrete defects are discernible. Bechtel Engineering approves the Use As Is disposition for the concrete as it has been placed in the structure. No remedial action is required.

Interim Field Report #5

The shield building concrete wall outside face is not within the plumb tolerance of 1 inch in any 25 feet. Reference Specification C-38.

Bechtel Engineering has reviewed the Interim Field Report and its attached plumb plots. Out of tolerance exceeds the 1 inch in 25 feet specified by 2-3/4 inches. The affect this has on the shield building structural integrity were found to be insignificant. Bechtel Engineering approves the Use As Is disposition for the structure and recommends that all interface work be adjusted to meet the as-built alignment of the structure.

Nonconformance Report #57

Pittsburg Testing Laboratory reports for cylinder 295, 301, and 302 shows that 156 cubic yards of Type II cement was used in place of Type I in the shield building wall between deck height 253 feet 4 inches and 256 feet 6-1/2 inches on 5-18-1971. Reference Specifications C-38 & C-26.

Bechtel Engineering approves the Use As Is disposition for the concrete based on the acceptable 28-day concrete compression tests as reported. Both the 28-day and 90-day concrete compression test results far exceed the specification of 4000 pounds per square foot compressive strength indicating that the change in cement type did not adversely affect the required strength characteristics.

Nonconformance Report #359

The concrete keyway at elevation 801 feet 6-1/2 inches at the top of the shield building wall second pour was constructed in an inverted position in order to not allow water to settle and freeze. Reference drawing C-109.

Bechtel Engineering approves the Use As Is disposition for the keyway based upon the fact that it does not change the structural analysis.

Nonconformance Report #382

Electrical blockouts at azimuth 82.8 degrees elevation 610 feet and 615 feet were not installed during shield building wall concrete placement. Reference drawing C-115. Concrete has been chipped out and reinforcing steel bars cut to place the 1 foot 3 inch square boxes required by the design drawings.

Bechtel Engineering approves the repair based on the fact it does not affect the structural integrity of the shield building. Place extra vertical reinforcing steel in the Purge line blockout to replace verticals cut by placement of the blockout. One horizontal bar of reinforcing steel will be disturbed on each face by each blockout with no affect on the structural integrity since many extra bars of horizontal reinforcing steel were added for the Purge line blockout at the same location.

Nonconformance Report #407

Approximately twenty #5 dowels were omitted or broken off attempting to bend them out from transfer tube penetrations through the shield building. Reference drawing C-113.

Bechtel Engineering approves the repair to drill holes, place #5 dowels, and grout with Embeco 636 for missing or broken dowels.

Nonconformance Report #415

Embedded plates for the station vent stack supports were not placed at locations on the shield building wall. Reference drawing C-112. All embedded plates are within a usable tolerance except for the embedded plate at elevation 625 feet 11-3/16 inches east of the station vent stack center line.

Bechtel Engineering approves the Use As Is disposition for all embedded plates except the one at elevation 625 feet 11-3/16 inches. For the embedded plate at elevation 625 feet 11-3/16 inches, cut plate which mounts to embed and weld as shown on sketches.

Nonconformance Report #451

Superseded by Nonconformance Report #479.

Nonconformance Report #457

The pipe sleeve for penetration #39 through the shield building must be placed at a fixed distance from the flued head anchor. The pipe sleeve flanges do not align with the concrete due to out of roundness of the shield building. Reference drawing C-115.

Bechtel Engineering approves the repair to place the sleeve in its proper location in relation to the flued head and adjust concrete and reinforcing steel to match as shown on sketches.

Nonconformance Report #474

Concrete was placed in the blockout for penetrations 33 and 40 at azimuth 237 degrees elevation 643 feet prior to approval of Nonconformance Report #451, therefore the flange cannot be moved as stated in the disposition. Reinforcing steel which extends into concrete was not placed with 3 inch clearance. Reference drawing C-115.

Bechtel Engineering approves the Use As Is disposition with sleeve placement in accordance with Nonconformance Report #479.

Nonconformance Report #479

The pipe sleeve for penetration #40 through the shield building must be placed at a fixed distance from the flued head anchor. The pipe sleeve flanges do not align with the concrete due to out of roundness of the shield building. Reference drawing C-115.

Bechtel Engineering approves the repair to place the sleeve in its proper location in relation to the flued head and adjust concrete and reinforcing steel to match as shown on sketches.

Nonconformance Report #602

The reglet in the shield building was not placed and maintained at a constant elevation. Counter flashing cannot be placed due to the waviness of the reglet. Reference drawing C-112.

Bechtel Engineering approves the repair that places a continuous saw cut in the shield building concrete at elevation 662.25 feet in lieu of the reglet shown on drawing C-112.

Nonconformance Report #743

Two #11 dowels are missing on the horizontal face of penetration #80 and the spacing of #8 and #10 vertical dowels along the top of penetration #80 exceeds the 20 inch maximum. Reference drawing C-115.

Bechtel Engineering approves the repair to place 2 grouted-in replacement dowels as shown on sketches, and a Use As Is disposition for the dowel spacing where jacking rods interfere since it will not affect the design or stress distribution of the shield building.

Nonconformance Report #772

The shield building reinforcing steel was installed at elevations beyond the construction tolerances. Reference Specification C-38 and drawing C-110.

Bechtel Engineering approves the Use As Is disposition for the reinforcing steel elevation deviations based upon the fact that it does not affect the integrity of the structure.

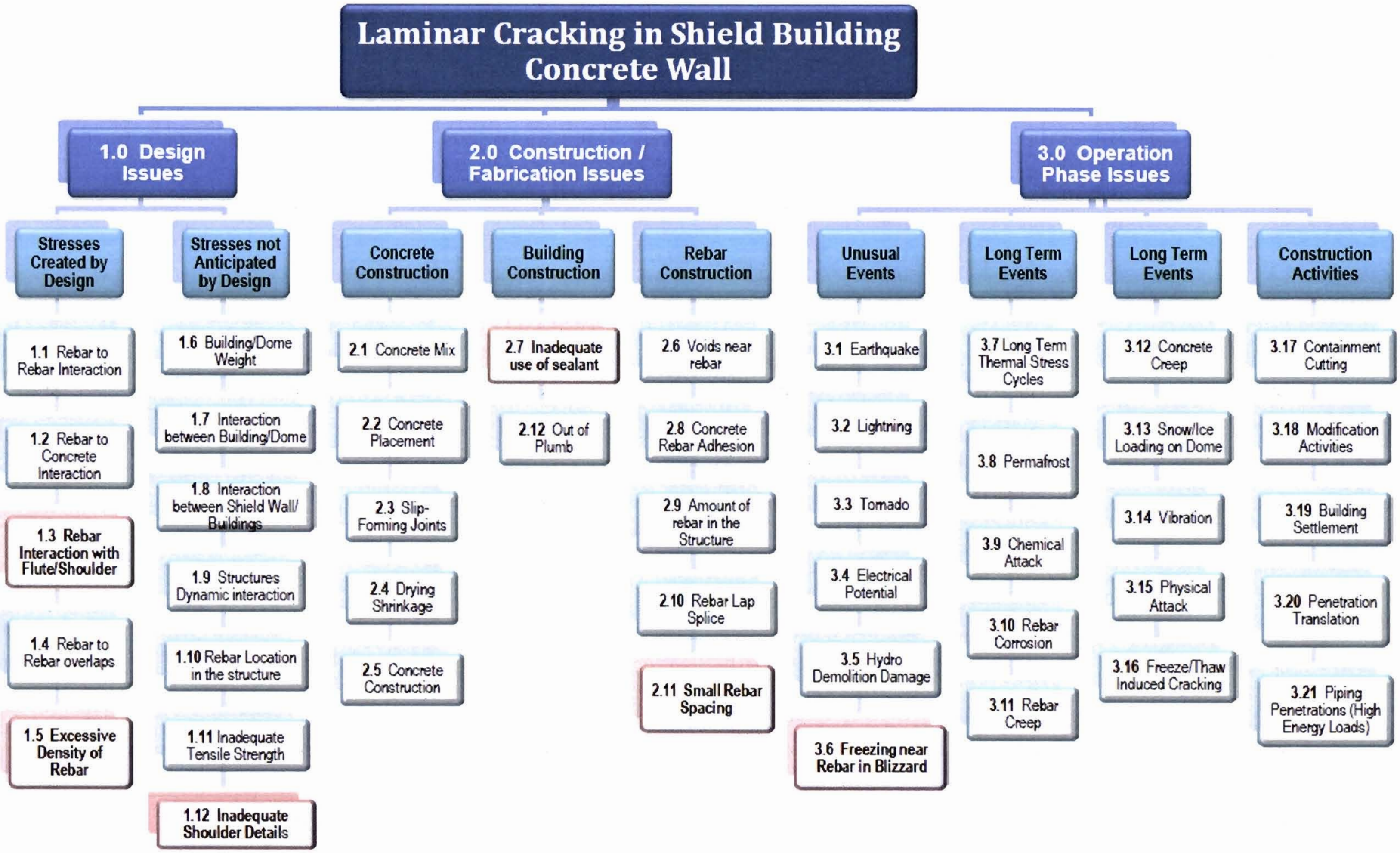
Material Rejection Report, 1-25-1971

6 cubic yards concrete mix ticket DB-00219 rejected by Fegles Power Service for 7-1/2 inch slump. Concrete disposed at burrow pit.

Material Rejection Report, 1-26-1971

6 cubic yards concrete mix ticket DB-00266 rejected by Fegles Power Service for 7 inch slump. Concrete disposed at burrow pit.

Attachment 11, Fault Tree



Attachment 12, Failure Modes Analysis

Failure Mode No. 1		Description: Design Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause	Summarize review of data collected to confirm or disprove cause
1.1) Rebar to rebar interaction	Initial laminar crack in the temporary access opening was located at a vertical to horizontal rebar interface within the architectural flute shoulder. None	Review design records Drawing C-110 Calculation C-CSS-099.20-054 Calculation C-CSS-099.20-056 Ground penetrating radar survey	Refuted. Rebar lap splice length is consistent or more conservative than ACI 318-63 requirements. Typically the stresses in the rebar and concrete are approximately ½ of the allowable values.
1.2) Rebar to concrete interaction	Initial laminar crack in the temporary access opening was located at a rebar to concrete interface within the architectural flute shoulder. None	Review design records Drawing C-110 Calculation C-CSS-099.20-054 Calculation C-CSS-099.20-056	Refuted. Rebar lap splice length is consistent or more conservative than ACI 318-63 requirements. Typically, the stresses in the rebar and concrete are approximately ½ of the allowable values.
1.3) Rebar interaction with flute / shoulder	Initial laminar crack in the temporary access opening was located at a rebar to concrete interface within the architectural flute shoulder. Similar laminar cracks were subsequently located at areas beyond the architectural flute shoulders such as near the top of the shield building, and adjacent to the main steam line penetration blockouts.	Review design records Drawing C-110 Drawing C-111A	**Causal Factor.** The architectural flute vertical rebar are not tied to the outside face rebar mat. The architectural flute horizontal rebar are tied to the main rebar only at the ends. There is an approximately 10 foot horizontal span in which the architectural flute shoulder concrete is not connected to the main rebar mat.

Failure Mode No. 1		Description: Design Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause	Summarize review of data collected to confirm or disprove cause
1.4) Rebar to rebar overlaps	Initial laminar crack in the temporary access opening was located at a rebar to rebar overlap area. Shield building designed for rebar lap splices.	Review construction photos Review design records Drawing C-110 Calculation C-CSS-099.20-054 Calculation C-CSS-099.20-056 Ground penetrating radar survey	Refuted. Rebar lap splice length is consistent or more conservative than ACI 318-63 requirements. Typically, the stresses in the rebar and concrete are approximately ½ of the allowable values.
1.5) Density of rebar	Initial laminar crack in the temporary access opening was located at an area with a high rebar density. Additional rebar was added at the construction opening and other similar blockout areas to compensate for the rebar interrupted by the opening.	Review construction photos Review design records Drawing C-110 Drawing C-112 Finite element analysis	**Causal Factor.** A rebar spacing sensitivity study established that a higher density of rebar could propagate laminar cracking beyond the architectural flute region with a given stress condition.
1.6) Building / dome weight	Some laminar cracks extended beyond the architectural flute shoulders such as those located near the top of shield building. Shield building designed for load from dome.	Review design records Drawing C-109 Drawing C-110 Drawing C-111A Calculation C-CSS-099.20-054	Refuted. The dead weight load from the building & dome is substantially less than the compressive strength of the concrete. Typically, the stresses in the rebar and concrete are approximately ½ of the allowable values.

Failure Mode No. 1		Description: Design Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause.	Summarize review of data collected to confirm or disprove cause.
1.7) Interaction between building / dome	Some laminar cracks extended beyond the architectural flute shoulders such as those located near the top of shield building. Shield building designed for load from dome.	Review design records Drawing C-109 Drawing C-110 Drawing C-111A Calculation C-CSS-099.20-054	Refuted. The stresses in the rebar and concrete are approximately ½ of the allowable values. Typically, the amount of rebar in the top 7-1/2 feet is double the ACI 307-69 requirements.
1.8) Interaction between wall / buildings	Some laminar cracks extended beyond the architectural flute shoulders such as those located adjacent to the main steam line penetration blockouts near the shield building and auxiliary building interface. Most laminar cracks were situated away from building interfaces. Shield building and auxiliary building designed for potential seismic loads.	Review design records Drawing C-110 Drawing C-111A Drawing C-200	Refuted. No relevant seismic activity. Shield building and auxiliary building both founded on bedrock. Buildings isolated by an expansion joint.
1.9) Structures dynamic interaction	Some laminar cracks extended beyond the architectural flute shoulders such as those located adjacent to the main steam line room penetration blockouts at shield building and auxiliary building interface. Most laminar cracks were situated away from building interfaces. Shield building and auxiliary building designed for potential seismic loads.	Review design records Drawing C-110 Drawing C-111A Drawing C-200	Refuted. No relevant seismic activity. Shield building and auxiliary building both founded on bedrock. Buildings isolated by an expansion joint.

Failure Mode No. 1		Description: Design Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause.	Summarize review of data collected to confirm or disprove cause.
1.10) Rebar location in structure	Operating experience documents indicate that rebar location in structure had previously been a problem area relevant to nuclear power plants. NRC IN 2008-17 & NUREG/CR-6927 None	Review design records Drawing C-110	Refuted. Rebar location was consistent with good engineering / fabrication practices.
1.11) Concrete tensile strength	Initial laminar crack in the temporary access opening was located in a uniform plane along the outer rebar mat under the architectural flute shoulder cross-section. None	Review design records Drawing C-111A Destructive examination of concrete cores	Refuted. Split tensile test results (>800 psi) were nearly double the ACI 318-89 value based upon the design minimum compressive strength.

Failure Mode No. 1		Description: Design Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause.	Summarize review of data collected to confirm or disprove cause.
1.12) Shoulder reinforcement detail	Initial laminar crack in the temporary access opening was located at a rebar to concrete interface in the architectural flute shoulder. Operating experience documents indicate that lack of radial reinforcement had previously been a problem area for concrete delamination at the Crystal River and Turkey Point nuclear power plants. NRC NUREG/CR-6927 Similar laminar cracks were subsequently located at areas beyond the architectural flute shoulders such as near the top of the shield building, and adjacent to the main steam line penetration blockouts.	Review design records Drawing C-110 Drawing C-111A	**Causal Factor.** The architectural flute vertical rebar are not tied to the outside face rebar mat. The architectural flute horizontal rebar are tied to the main rebar only at the ends. There is an approximately 10 foot horizontal span in which the architectural flute shoulder concrete is not connected to the main rebar mat.

<table>
<tr><th colspan="2">Failure Mode No. 2</th><th colspan="2">Description: Construction / Fabrication Issues</th></tr>
<tr><td rowspan="2">POSSIBLE CAUSE(S)</td><td>Existing data that supports this as the cause.</td><td rowspan="2">Data required to confirm or disprove this cause</td><td rowspan="2">Summarize review of data collected to confirm or disprove cause</td></tr>
<tr><td>Existing data that tends to disprove this as the cause.</td></tr>
<tr><td rowspan="2">2.1)
Concrete mix</td><td>Operating experience documents indicate that concrete mix had previously been a problem area relevant to nuclear power plants. NRC IN 2008-17 & INPO IER L4-11-4.</td><td rowspan="2">Review design and construction records
Specifications C-26 & C-38
Destructive examination of concrete cores</td><td rowspan="2">Refuted.
The concrete mix achieved material property requirements for aggregate, cement type, concrete strength, durability, air content, and workability.
Nonconformances were localized.</td></tr>
<tr><td>Pittsburg Testing Laboratory independently verified each mix ticket.</td></tr>
<tr><td rowspan="2">2.2)
Concrete placement</td><td>Operating experience documents indicate that concrete placement had previously been a problem area relevant to nuclear power plants. NRC IN 2008-17 & INPO IER L4-11-4.</td><td rowspan="2">Review design and construction records
Specifications C-26 & C-38</td><td rowspan="2">Refuted.
The concrete placement achieved material property requirements for segregation, temperature, water content, cover, curing and placing.
Nonconformances were localized.</td></tr>
<tr><td>None</td></tr>
<tr><td rowspan="2">2.3)
Slip-form joints</td><td>Cold joints between hardened and fresh concrete occurred between the two major pours and other times such as weekends.</td><td rowspan="2">Review design, construction, and operation records
Drawing C-111A</td><td rowspan="2">Refuted.
The design specification addresses how to continue placement beyond cold joints.</td></tr>
<tr><td>The laminar cracks were oriented perpendicular to the slip-form joints and also mostly on the southern exposure versus circumferential.
Visual examination of the shield building exterior did not observe surface distress resulting from cold joints allowing moisture movement.</td></tr>
</table>

Failure Mode No. 2		Description: Construction / Fabrication Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause	Summarize review of data collected to confirm or disprove cause
2.4) Drying shrinkage	Operating experience documents indicate that concrete shrinkage had previously been a problem area relevant to nuclear power plants. NRC NUREG/CR-6927 Visual examination of shield building exterior concrete did not observe active surface distress resulting from shrinkage allowing moisture movement.	Review operation records	Refuted. Visual examination of shield building exterior concrete did not observe active surface distress resulting from shrinkage allowing moisture movement.
2.5) Concrete construction	Operating experience documents indicate that concrete construction had previously been a problem area relevant to nuclear power plants. NRC IN 2008-17 & INPO IER L4-11-4. None	Review design and construction records Specifications C-26 & C-38 Destructive examination of concrete cores	Refuted. The concrete construction achieved process requirements for vibration, pour timing, joints, forms, slip-form speed, and jacking rod configuration. The concrete material properties were acceptable.
2.6) Voids near rebar	Operating experience documents indicate that voids near rebar had previously been a problem area relevant to nuclear power plants. NRC NUREG/CR-6927 Visual examination of shield building rebar at temporary access opening did not veal any substantial difference in voids adjacent to rebar or away from rebar	Review operation records Destructive examination of concrete cores	Refuted. Petrographic examination found no substantial difference in voids adjacent to rebar or away from rebar.

Failure Mode No. 2		Description: Construction / Fabrication Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause	Summarize review of data collected to confirm or disprove cause
2.7) Concrete sealant	Industry documents indicate that moisture intrusion into concrete had previously been a problem area above grade due to hydrostatic pressure, water vapor gradient, capillary action, wind-driven rain, or any combination of these forces. ACI 515.1 R-79. None	Review design and construction records Specification C-38 Destructive examination of concrete cores Visual examination of shield building exterior	**Causal Factor.** Construction records indicate a sealant was applied to protect the concrete only during curing. There was no sealant specified to mitigate moisture penetration. Destructive examination of concrete cores identified that the surface zone is not water-repellant, and the presence of deposits in air voids typically suggests long-term exposure to moisture migrating through the concrete.
2.8) Concrete to rebar adhesion	Initial laminar crack in the temporary access opening was located at a rebar to concrete interface within the architectural flute shoulder. Visual examination of shield building rebar at temporary access opening identified numerous examples of concrete strongly adhered to rebar.	Review design and operation records Drawing C-111A	Refuted. Reinforced concrete design relies upon rebar deformation versus adhesion to transfer the stresses.

Failure Mode No. 2		Description: Construction / Fabrication Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause	Summarize review of data collected to confirm or disprove cause
2.9) Amount of rebar in structure	Operating experience documents indicate that the amount of rebar had previously been a problem area relevant to nuclear power plants. NRC IN 2008-17 & INPO IER L4-11-4. None	Review design records Drawing C-110 Drawing C-111 Drawing C-112 Calculation C-CSS-099.20-054 Calculation C-CSS-099.20-056	Refuted. The amount of rebar was less than the global density for brittle fracture. The amount of rebar was greater than the strength needed for the required loads.
2.10) Rebar lap splice	Initial laminar crack in temporary construction opening was located at rebar to rebar overlap area None	Review design records Drawing C-110 Calculation C-CSS-099.20-054 Calculation C-CSS-099.20-056 Ground penetrating radar survey	Refuted. Rebar lap splice length is consistent or more conservative than ACI 318-63 requirements. Typically, the stresses in the rebar and concrete are approximately ½ of the allowable values.

Failure Mode No. 2		Description: Construction / Fabrication Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove this cause	Summarize review of data collected to confirm or disprove cause
2.11) Small rebar spacing	Initial laminar crack in the temporary access opening was located at an area with a high rebar density. Additional rebar was added at the construction opening and other similar blockout areas to compensate for the rebar interrupted by the opening.	Review construction photos Review design records Drawing C-110 Drawing C-112 Finite element analysis	**Causal Factor.** A rebar spacing sensitivity study established that a higher density of rebar could propagate laminar cracking beyond the architectural flute region with a given stress condition.
2.12) Plumb	Plumb tolerance was exceeded during the above-grade pour. Plumb tolerance issues oriented different than the laminar cracking locations.	Review construction records	Refuted. The effect of the out of tolerance plumb was insignificant to structure integrity.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.1) Earthquake	A classifiable event from seismic activity occurred on March 5, 1986. Shield building was designed for potential adverse environmental conditions including seismic loads.	Review operational records Unit log Event number 3837	Refuted. No relevant seismic activity. Seismic system actuation attributed to movement of heavy equipment near trigger.
3.2) Lightning	A potential lightning strike of the shield building occurred on May 10, 1995. Shield building was designed for potential adverse environmental conditions including lightning strikes.	Review operational records Drawing E-401 Problem report 1995-0395	Refuted. No actual lightning strike damage found. Sufficiently grounded at the dome.
3.3) Tornado	A classifiable event from a tornado occurred on June 24, 1998. Shield building was designed for potential adverse environmental conditions including tornado loads.	Review operational records Licensee Event Report 98-006 Finite element analysis	Refuted Insufficient radial stress from tornado loads to initiate or propagate the laminar cracks.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.4) Electrical potential	Initial laminar crack in the temporary access opening was located in an area observed with corrosion and rust stains on the outer mat of rebar. Visual examination of the shield building rebar at temporary access opening did not observe excessive rebar corrosion or material loss.	Destructive examination of concrete cores	Refuted. Destructive examination of concrete cores found no accelerated rebar corrosion produced from galvanic action due to an unbalance in electric potential.
3.5) Hydrodemolition	Initial laminar crack in the temporary access opening was located at the boundary of the hydrodemolition. Similar laminar cracks were subsequently located at areas beyond the hydrodemolition boundary such as near the top of the shield building, and adjacent to the main steam line penetration blockouts.	Review operating experience Drawing C-111A	Refuted. Similar laminar cracks were subsequently located at areas beyond the hydrodemolition boundary such as near the top of the shield building, and adjacent to the main steam line penetration blockouts.
3.6) Freezing of water near rebar in blizzard	A severe blizzard occurred surrounding the facility on January 26 & 27, 1978. Shield building was designed for potential adverse environmental conditions including wind and thermal loads.	Review operational records Unit log Licensee Event Report 78-017 Finite element analysis	**Causal Factor.** The wind load moisture intrusion into the unsealed concrete followed by the thermal load from the moisture freezing and expanding were sufficient to produce a radial stress to initiate the laminar cracks

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.7) Long term thermal stress cycles	Most laminar cracks were oriented on the southern exposure of the shield building exterior. Some laminar cracks were located at main steam line room interface with the shield building at the piping penetration blockouts. Operating experience documents indicate that prolonged exposure of concrete to elevated temperatures results in decreased elastic modulus and compressive rupture strength. EPRI-1020932. A general threshold limit for temperature exposure degradation of concrete is approximately 95 degrees Centigrade / 203 degrees Fahrenheit.	Review design and operational records Drawing C-111A Thermal imaging Destructive examination of concrete cores Finite element analysis	Refuted. No micro-cracks evident. Insufficient radial stress from thermal loads (<1/2 tensile strength) to initiate or propagate the laminar cracks. Insufficient thermal and shrinkage strains to cause cracking. Insufficient temperature (<150 degrees Fahrenheit) for temperature exposure degradation adjacent to the main steam line penetration blockouts. No actual fires adjacent to the shield building.
3.8) Permafrost	Permafrost produces cracks parallel to the ground surface Laminar cracks are located above grade.	Review design and operational records Drawing C-110 Drawing C-111A	Refuted. The mean air temperature near the facility is greater than freezing necessary for permafrost.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.9) Chemical attack	Operating experience documents indicate that chemical attack had previously been a problem area relevant to nuclear power plants. NRC NUREG/CR-6927, Information Notice 2011-20, and EPRI-1020932. Leaching & efflorescence - water passing through cracks dissolves the constituents in cement paste. Sulfate attack - magnesium and sulfates present in soil, groundwater, and acid rain can react with the cement paste and cause swelling and irregular cracking. Ettringite formation – reaction of sulfate with calcium aluminates resulting in expansion and cracking. Carbonation - carbon dioxide in the atmosphere reacts with calcium hydroxide or other lime-bearing compounds and results in a reduction in pH of the cement paste which has the potential to cause degradation to embedded steel reinforcement. Alkali-aggregate reaction - reaction of alkali ions in cement with silica mineral aggregates forms a gel that expands when it comes into contact with water and manifests as small surface cracks in an irregular pattern. Acids and bases - acidic aqueous solutions attack the cement paste leading to increased porosity. High concentration bases can disintegrate concrete. Visual examination of the shield building exterior did not observe excessive chemical attack.	Review operation records Destructive examination of concrete cores	Refuted. Destructive examination of concrete cores found no evidence of chemical attack producing the laminar cracking. Only inconsequential amounts of ettringite formation and carbonation were identified.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.10) Rebar corrosion	Initial laminar crack in the temporary access opening was located in an area observed with corrosion and rust stains on the outer mat of rebar. Visual examination of the shield building exterior did not observe excessive cracking, staining or spalling. Visual examination of the shield building exterior did not observe excessive rebar corrosion or material loss.	Review design, construction, and operational records Destructive examination of concrete cores	Refuted. Sufficient barriers for minimizing rebar corrosion were established including low water permeability concrete mix design, and adequate rebar cover. Destructive examination of concrete cores found an inconsequential carbonation depth.
3.11) Rebar creep	None None	Finite element analysis	Refuted. Insufficient radial stress from thermal loads in the concrete to affect the creep threshold for rebar (>30,000 psi).
3.12) Concrete creep	Strain that accumulates due to dead or live static loads over long periods of time as a function of the loading magnitude and history, environment, and material properties of the concrete. EPRI-1020932. Creep in those concrete structures not pre-stressed is not considered to be a significant degradation mechanism, because cracking is generally not sufficient to expose the steel reinforcement and has a minor effect on structural integrity. EPRI-1020932.	Destructive examination of concrete cores	Refuted. The measured concrete creep coefficient is in the normal range and not sufficient to cause laminar cracks.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.13) Snow & ice loading to dome	Some laminar cracks extended beyond the architectural flute shoulders such as those located near the top of the shield building. Shield building designed for live and dead loads from dome.	Review design records Drawing C-109 Drawing C-110 Drawing C-111A Calculation C-CSS-099.20-054	Refuted. The live & dead weight load from the building & dome is substantially less than the compressive strength of the concrete.
3.14) Vibration / fatigue	Operating experience documents indicate that cyclic loading in concrete structures from vibration or fatigue initiates as microcracks in the cement paste adjacent to aggregate particles, reinforcing steel, or stress concentrations which in the later stages can manifest itself as large, structurally significant cracks. EPRI-1020932. Operating experience documents indicate that fatigue failure of concrete is unusual because of its good fatigue resistance. IAEA TECDOC-1025.	Review design and operation records Destructive examination of concrete cores.	Refuted. No rotating equipment located on or supported by the shield building. No adverse vibration identified by monitoring major rotating equipment inside containment. Destructive examination of concrete cores found no microcracks related to vibration / mechanical fatigue.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.15) Physical attack	Operating experience documents indicate that physical attack had previously been a problem area relevant to nuclear power plants. NRC NUREG/CR-6927 and EPRI-1020932. Abrasion or cavitation – loss of concrete material on concrete surfaces exposed to fluid flow, impingement, or negative surface pressure. Irradiation - dehydration of concrete and changes in mechanical properties when exposed to prolonged or very high doses. Salt crystallization – movement of salt solution by capillary action generating expansive forces that result in the physical breakdown of the concrete. Visual examination of the shield building exterior did not observe excessive physical attack.	Review of design, construction, and operation records Destructive examination of concrete cores.	Refuted. Destructive examination of concrete cores found no evidence of physical attack producing the laminar cracking.
3.16) Freeze / thaw	Operating experience documents indicate that the freeze / thaw cycle of entrained water has demonstrated spalling at the surface and localized internal cracking. EPRI-1020932. The freeze / thaw cycle damage typically occurs in concrete in contact with water, such as on flat horizontal surfaces. Visual examination of the shield building exterior did not observe excessive surface deterioration.	Review of operation records Destructive examination of concrete cores.	Refuted. Destructive examination of concrete cores found no microcracks related to freeze / thaw thermal fatigue, or freeze / thaw deterioration at the core surface.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.17) Containment cutting	Initial laminar crack in the temporary access opening was located adjacent to a previous temporary access opening and the original construction opening. Similar laminar cracks were subsequently located at areas beyond the hydrodemolition boundary such as near the top of the shield building, and adjacent to the main steam line penetration blockouts.	Review operating experience Drawing C-111A	Refuted. No previous operating experience with laminar cracking caused by hydrodemolition. Literature review concluded that hydrodemolition poses less damage to adjacent material than mechanical impact alternatives.
3.18) Modification activities	Initial laminar crack in the temporary access opening was located adjacent to a previous temporary access opening. Similar laminar cracks were subsequently located at areas beyond the hydrodemolition boundary such as near the top of the shield building, and adjacent to the main steam line penetration blockouts.	Review operating experience Drawing C-111A	Refuted. Similar laminar cracks were subsequently located at areas beyond the hydrodemolition boundary such as near the top of the shield building, and adjacent to the main steam line penetration blockouts.

Failure Mode No. 3		Description: Operational Issues	
POSSIBLE CAUSE(S)	Existing data that supports this as the cause. Existing data that tends to disprove this as the cause.	Data required to confirm or disprove cause	Summarize review of data collected to confirm or disprove cause
3.19) Building settlement	Operating experience documents indicate that structures have a tendency to settle during construction and early life that can lead to concrete cracking. IAEA TECDOC-1025. Minimal settlement is expected for foundations situated on bedrock. No laminar cracks were located adjacent to the inner face rebar mat. Visual examination of the shield building exterior did not observe excessive surface distress.	Review of design and operation records Drawing C-100	Refuted. Shield building founded on bedrock.
3.20) Penetration translational stress	Some laminar cracks extended beyond the architectural flute shoulders such as those located adjacent to the main steam line room penetration blockouts at shield building and auxiliary building interface. Most laminar cracks were situated away from penetration blockouts.	Review of design records Drawing C-111A Drawing M-284A Drawing M-284B	Refuted. The high energy piping penetration loads are structurally isolated from the shield building.
3.21) Piping penetration loads	Some laminar cracks extended beyond the architectural flute shoulders such as those located adjacent to the main steam line penetration blockouts at shield building and auxiliary building interface. Most laminar cracks were situated away from penetration blockouts.	Review of design records Drawing C-111A Drawing M-284A Drawing M-284B	Refuted. The high energy piping penetration loads are structurally isolated from the shield building.

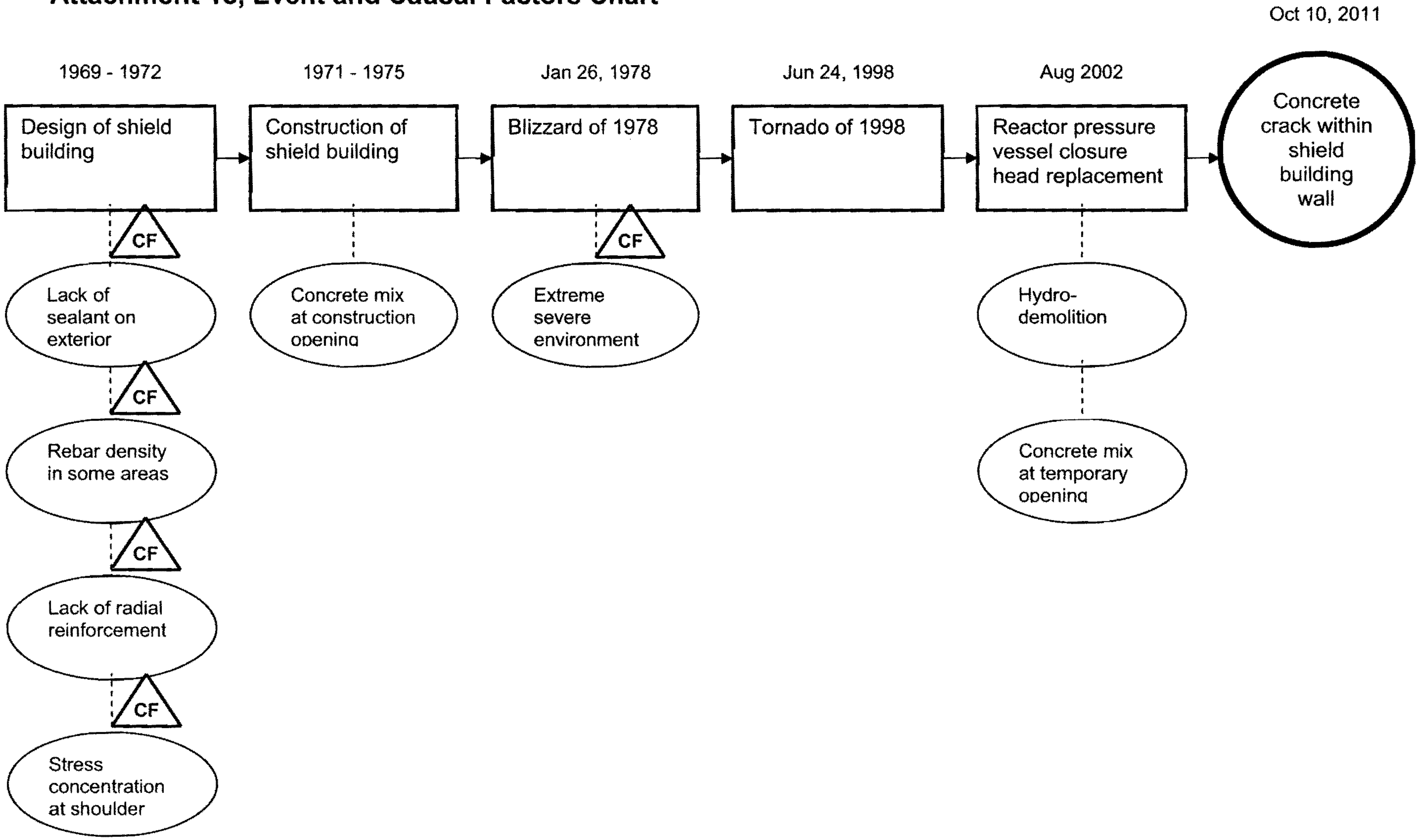
Attachment 13, Change Analysis

Change Analysis		
Current Condition	**Previous Condition**	**Impact**
Since 10/1973 the dome has been in place on the shield building.	Prior to installation of the shield building dome, the walls had reinforcing steel / jacking rods that were exposed to environmental conditions	Potential impact due to a broad pathway for moisture ingress to facilitate reinforcing steel corrosion or freeze / thaw damage.
Since 12/1975 the original construction opening has been closed with a different mix of concrete having smaller aggregate than the shield building wall.	Prior to closure of the original construction opening, the perimeter reinforcing steel were exposed to environmental conditions	Insignificant impact other than both a potential local pathway form moisture ingress to facilitate reinforcing steel corrosion or freeze / thaw damage, and a different concrete mix with smaller aggregate for closure of the original construction opening.
Since 08/1977 the reactor has been critical except for refueling or maintenance shutdowns.	Prior to start-up testing and reactor criticality there was lesser heat and radiation on the annulus side of the shield building.	Insignificant impact as the radiation source on the shield building during normal operation term is minimal, and the differential temperature for thermal cycling is less severe than during operation.
Since 09/2002 a temporary access opening has been cut into the shield building wall by hydrodemolition and closed with a different mix of concrete having smaller aggregate than the shield building wall.	Prior to creating and closing the temporary access opening, the concrete of the original construction opening had not been disturbed.	Insignificant impact other than a different concrete mix with smaller aggregate for closure of the temporary access opening.

Attachment 14, Barrier Analysis

Barrier Analysis		
Consequence / Adverse Effect	**Barrier that should have Precluded Consequence**	**Barrier Assessment (Why the Barrier Failed)**
Lack of sealant on exterior	Design basis	Not in specification
Rebar density in some areas	Design basis	Not concerned with amount of rebar
Lack of radial reinforcement	Design basis	Not concerned with architectural elements
Stress concentration behind flute	Design basis	Not concerned with architectural elements

Attachment 15, Event and Causal Factors Chart



Attachment 16, Generic Implications Matrix

Item	Original Scope	Extent of Condition
Program / Process	Hydrodemolition of temporary access opening for replacement of reactor pressure vessel closure head	Slip-form construction of right cylindrical structures
Plant / System / Structure / Component	Davis-Besse shield building	Nuclear reactor containment shield buildings Chimneys / Stacks Storage tanks / Silos Cooling towers
Organization	Supplemental personnel	Construction Management / Oversight Architectural Engineer Civil Engineer
Environment	Architectural flutes	Outer reinforcing steel mat Southwestern exposure
People / Group	American Hydro	Fegles Power Service / Chicago Bridge and Iron Bechtel

Attachment 17, Corrective Action Matrix

Corrective Action or Notification Number	Cause	Cause Code	Corrective Action Description	Corrective Action Type	Group Responsible	Due Date	Safety Precedence Sequence
CA-2011-03346-1	Extent of Condition CA1	T22	Additional Examination of the Shield Building Exterior Wall	Corrective	Site Projects	07/15/2012	N/A
CA-2011-03346-2	Extent of Condition CA2	T22	Issue Engineering Change Package for Additional Shield Building Core Bores	Corrective	Design Engineering	04/30/2012	N/A
CA-2011-03346-3	Direct Cause CA1	T22	Testing Program to Investigate the Steel Reinforcement Capacity Adjacent to Structural Discontinuities	Corrective	Design Engineering	08/01/2012	N/A
CA-2011-03346-4	Direct Cause CA2	T22	Engineering Plan to Re-Establish Design & Licensing Basis for Shield Building	Corrective	Design Engineering	12/01/2012	N/A
CA-2011-03346-5	Direct Cause CA3	T22	Issue a Site Specific Procedure for the Long-Term Monitoring of the Shield Building Laminar Cracking	Corrective	Design Engineering	07/11/2012	4 - procedure
CA-2011-03346-6	Root Cause CA1	DA1D	Issue Engineering Change Package for a Shield Building Exterior Sealant System	Preventive	Design Engineering	05/01/2012	1 – design for minimum hazard
CA-2011-03346-7	Root Cause CA2	DA1D	Implement Engineering Change Package for a Shield Building Exterior Sealant System	Preventive	Site Projects	10/01/2012	1 – design for minimum hazard
CA-2011-03346-8	Root Cause CA3	DA1D	Update Inspection Procedure to Include Shield Building Exterior Sealant System	Corrective	Design Engineering	10/01/2012	4 - procedure
CA-2011-03346-9	CAL Commitment CA1	N/A	Root Cause Report Submittal	Corrective	Design Engineering	03/02/2012	N/A
CA-2011-03346-10	CAL Commitment CA2	N/A	Examine Four Un-Cracked Core Bores Following Restart	Corrective	Design Engineering	03/05/2012	N/A
CA-2011-03346-11	CAL Commitment CA3	N/A	Main Steam Line Room New Core Bore & Examination Following Restart	Corrective	Site Projects	03/05/2012	N/A

Corrective Action or Notification Number	Cause	Cause Code	Corrective Action Description	Corrective Action Type	Group Responsible	Due Date	Safety Precedence Sequence
CA-2011-03346-12	CAL Commitment CA4	N/A	Examine Six Un-Cracked Core Bores in 17RFO	Corrective	Design Engineering	06/15/2012	N/A
CA-2011-03346-13	CAL Commitment CA5	N/A	Examine Three Crack Interface Core Bores in 17RFO	Corrective	Design Engineering	06/15/2012	N/A