Davis-BesseNPEm Resource

From: Sanchez Santiago, Elba
Sent: Thursday, November 17, 2011 7:33 PM
To: Murphy, Martin; Lupold, Timothy; Cameron, Jamnes; Wilson, Adam; Kimble, Daniel; Rutkowski, John; Hills, David; Rezai, Ali; Gonzalez, Hipolito; Thorp, John; Haskell, Russell; Nolan, Ryan; Mahoney, Michael; Hernandez, Pete; Mitlyng, Viktoria; Chandrathil, Prema; Neurauter, James; Cardona-Morales, Pedro; Briley, Thomas; CuadradoDeJesus, Samuel; Zimmerman, Jacob; Thomas, George; Hoang, Dan; Logaras, Harral; Barker, Allan; Auluck, Rajender; Sheikh, Abdul; Lehman, Bryce; Morey, Dennis; Burza, Justine; Snyder, Amy; Wiebe, Joel; Bozga, John; Meghani, Vijay; Stone, AnnMarie; Smagacz, Phillip; Davis-BesseHearingFile Resource; Riley (OCA), Timothy; Jessup, William; Graves, Herman; Pires, Jose; Hogan, Rosemary; Case, Michael; Richards, Stuart; Manoly, Kamal; Orth, Steven; 'meghanivijay@hotmail.com'; dekimble@bex.net; Sakai, Stacie; abdul shaikh
Cc: OBrien, Kenneth; Shear, Gary; West, Steven; Reynolds, Steven; Hiland, Patrick
Subject: FW: DB Shield Building Investigation and Technical Summary Report
Attachments: Shield Building Investigation and Technical Summary Report .pdf

All,

Attached is the technical summary provided by the licensee on the cracks identified in the shield building. This document also includes the assumptions and inputs that are being used in their current evaluation. I will be uploading this document to the SharePoint site, under the current evaluations folder. If possible, we intend to have an internal conference call tomorrow afternoon to discuss this report. Thank you all for the dedication and effort you are putting into the review of this issue. Feel free to contact me if you have any questions.

Thanks,

Elba M. Sanchez Santiago
Reactor Engineer
RIII/ DRS/ EB1
630-829-9715

Hearing Identifier: Davis_BesseLicenseRenewal_Saf_NonPublic
Email Number: 3597

Mail Envelope Properties (ED8A9A7B222CD84E8534B59A536552B6135C66A7AC)

Subject: FW: DB Shield Building Investigation and Technical Summary Report
Sent Date: 11/17/2011 7:32:54 PM
Received Date: 11/17/2011 7:33:02 PM
From: Sanchez Santiago, Elba

Created By: Elba.SanchezSantiago@nrc.gov

Recipients:
"OBrien, Kenneth" <Kenneth.OBrien@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:14:00 AM
"Shear, Gary" <Gary.Shear@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:16:00 AM
"West, Steven" <Steven.West@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:16:00 AM
"Reynolds, Steven" <Steven.Reynolds@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:36:00 AM
"Hiland, Patrick" <Patrick.Hiland@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Murphy, Martin" <Martin.Murphy@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:41:00 AM
"Lupold, Timothy" <Timothy.Lupold@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Cameron, Jamnes" <Jamnes.Cameron@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Wilson, Adam" <Adam.Wilson@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Kimble, Daniel" <Daniel.Kimble@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Rutkowski, John" <John.Rutkowski@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:16:00 AM
"Hills, David" <David.Hills@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:15:00 AM
"Rezai, Ali" <Ali.Rezai@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Gonzalez, Hipolito" <Hipolito.Gonzalez@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Thorp, John" <John.Thorp@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Haskell, Russell" <Russell.Haskell@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:15:00 AM
"Nolan, Ryan" <Ryan.Nolan@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Mahoney, Michael" <Michael.Mahoney@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:14:00 AM
"Hernandez, Pete" <Pete.Hernandez@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:14:00 AM
"Mitlyng, Viktoria" <Viktoria.Mitlyng@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:16:00 AM
"Chandrathil, Prema" <Prema.Chandrathil@nrc.gov>

Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Neurauter, James" <James.Neurauter@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:47:00 AM
"Cardona-Morales, Pedro" <Pedro.Cardona-Morales@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Briley, Thomas" <Thomas.Briley@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"CuadradoDeJesus, Samuel" <Samuel.CuadradoDeJesus@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Zimmerman, Jacob" <Jacob.Zimmerman@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Thomas, George" <George.Thomas2@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Hoang, Dan" <Dan.Hoang@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 9:57:00 AM
"Logaras, Harral" <Harral.Logaras@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Barker, Allan" <Allan.Barker@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Auluck, Rajender" <Rajender.Auluck@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Sheikh, Abdul" <Abdul.Sheikh@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 10:52:00 AM
"Lehman, Bryce" <Bryce.Lehman@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Morey, Dennis" <Dennis.Morey@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Burza, Justine" <Justine.Burza@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Snyder, Amy" <Amy.Snyder@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Wiebe, Joel" <Joel.Wiebe@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Bozga, John" <John.Bozga@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Meghani, Vijay" <Vijay.Meghani@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Stone, AnnMarie" <AnnMarie.Stone@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Smagacz, Phillip" <Phillip.Smagacz@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Davis-BesseHearingFile Resource" <Davis-BesseHearingFile.Resource@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Riley (OCA), Timothy" <Timothy.RileyOCA@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:15:00 AM
"Jessup, William" <William.Jessup@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Graves, Herman" <Herman.Graves@nrc.gov>
Tracking Status:: Response: None : 11/17/2011 8:24:00 AM
"Pires, Jose" <Jose.Pires@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Hogan, Rosemary" <Rosemary.Hogan@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Case, Michael" <Michael.Case@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Richards, Stuart" <Stuart.Richards@nrc.gov>

Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Manoly, Kamal" <Kamal.Manoly@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"Orth, Steven" <Steven.Orth@nrc.gov>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"'meghanivijay@hotmail.com'" <meghanivijay@hotmail.com>
Tracking Status:: Response: None : 10/22/1602 6:18:00 PM
"dekimble@bex.net" <dekimble@bex.net>
Tracking Status: None
"Sakai, Stacie" <Stacie.Sakai@nrc.gov>
Tracking Status: None
"abdul shaikh" <abdulhaisheikh@yahoo.com>
Tracking Status: None

Post Office: R3CLSTR01.nrc.gov

Files	**Size**	**Date & Time**
MESSAGE	679	11/17/2011 7:33:02 PM
Shield Building Investigation and Technical Summary Report .pdf	557431	

Options
Priority: Standard
Return Notification: No
Reply Requested: No
Sensitivity: Normal
Expiration Date:
Recipients Received:

Davis-Besse Shield Building

Investigation and Technical Summary

November 17, 2011 Rev 0

Prepared by: R.N. BAIR R. N. Bair

Reviewed by: Jon G. Hook Jon G. Hook

Approved by: KW Byrd KW Byrd

Table of Contents

Figures

1.0 Condition

On October 10, 2011, a laminar crack was found in the architectural flute shoulder area of the opening being cut through the Shield Building cylindrical wall for replacement of the reactor vessel closure head (RVCH). The crack was found to be on the vertical side of the opening (left side, looking from the outside), generally along the main reinforcing steel of the cylinder, and extending to across the top (approx 6 feet) and across the bottom (approximately 4 feet) of the opening. After some minor manual chipping along the edges, the crack indication along the left and bottom edges essentially disappeared.

In the area above the opening, a portion of the area was chipped upward approximately 23 inches which showed that the overall crack length reduced to approximately 5-feet long. The crack appeared to be tight and confined to the architectural fluted area.

Based on the observed conditions, the crack is considered a circumferential laminar tear and not a radial through-thickness directional crack. Condition Report 2011-03346 (Ref. 1) was initiated to identify the issue.

To further investigate the condition, an investigation program of Impulse Response testing, and core bore verification was initiated. This program revealed similar cracking in each flute shoulder inspected. Cracking was also identified outside of the flute shoulders at the top of the Shield Building wall and local cracking around corners of blockouts for steam line Penetrations 39 and 40 was also detected. Condition Reports 2011-04648 (Ref. 7), and 2011-04402 (Ref. 5) were initiated and titled “Shield Building IR Indications above El 780 feet” and “Fractured Concrete found at 17M near MSL Penetrations” respectively to address these conditions. A complete Condition Assessment is provided in Section 4 of this document.

2.0 Shield Building Description

The Shield Building is a safety related free standing cylindrical shell structure with eight (8) “architectural flute areas” as identified on DBNPS Drawing C-0110 (Ref. 8). During further discussions within this document, each “fluted area” will be addressed as two built up, or thickened “shoulders.” The groove between the shoulders will be addressed as the “flute.”

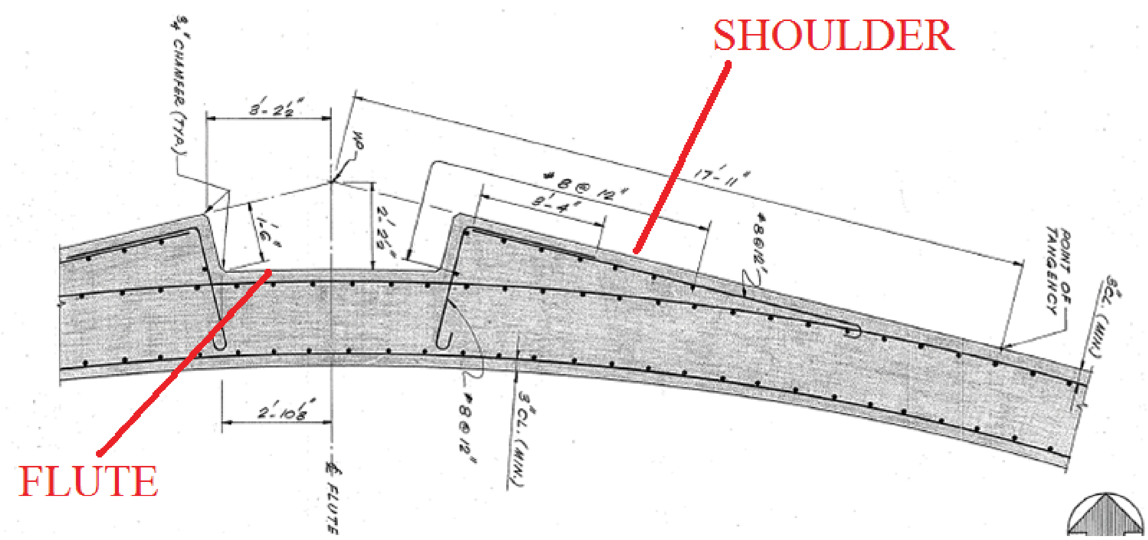

Figure 01 – Flute / Shoulder Geometry

A standard was adopted to number the flutes 1 through 8 clockwise with the first flute at azimuth 22.5^0 of the Shield Building. Likewise, the areas adjacent to the flutes were referred to as the flute shoulder areas and numbered 1 through 16 clockwise from the zero degree azimuth of the Shield Building. See Figure 01 and drawing C-0111A (Ref. 9) for reference. The zero degree azimuth is equal to plant reference north.

The cylindrical shell region of the Shield Building has a design thickness of 30 inches, with 3 inches minimum clear cover specified for exterior and interior reinforcement. The shoulder regions have a thickness of 18 inches and length of 17 feet 11 inches from the flute centerline to point of tangency at the cylindrical shell. The specified construction tolerance on wall thickness was +1.0 inch or – ¼ inch (Ref. 10).

The concrete for the Shield Building was specified as Class C-2, with a compressive strength of 4,000 psi at 28 days. Reinforcing steel was specified as ASTM A-615-68, Grade 60 (Ref. 8).

The Shield Building design does not consider the flute shoulder area as providing additional structural capacity beyond that provided by the cylindrical shell or dome. From a loading standpoint on the structure, the Shield Building has been evaluated considering the flute shoulder area as an additional dead load.

Reinforcing within the cylindrical shell consists of meridian (vertical) and circumferential (hoop) reinforcing bars forming a grid on both faces of the shell. On the outside face of the shell, the hoop reinforcing is located outside the vertical bars. The same arrangement is true for the inside face reinforcing (i.e. the hoop bars are the inner most reinforcing layer. Reinforcing for the shoulders consists of vertical bars and horizontal ties conforming to the profile of the shoulder.

These horizontal ties are anchored into the shell at both ends. Reinforcing details for the Shield Building can be found on DBNPS Drawing C-110 (Ref. 8).

3.0 Design / Licensing Basis

The Updated Safety Analysis Report (USAR) (Ref. 11) Section 3.8.2.2.2 describes the design basis of the Shield Building as:

- Biological shielding
- Providing for the controlled release of the annulus atmosphere under accident conditions
- Provide environmental protection for the Containment Vessel (tornado wind & differential pressure, missiles, etc.

USAR Section 3.8.2.2.4 & 3.8.2.3.4 describes the load combinations for the Shield Building. Per the USAR, the most severe load combination per the ultimate strength design method is $(D + L + E' + T_{A)}$ while $(D + T_O + E)$ is the most severe load combination per the working stress design method.

Where:

D = Dead load;
L = Live load
E' = Maximum Possible Earthquake
T_A = Temperature at Accident
T_O = Temperature at Operating Conditions
E = Maximum Probable Earthquake

4.0 Condition Assessment

4.1 Initial Investigation

The condition was initially identified on October 10 when cracking was reported along the left side of the new construction opening in the Shield Building during hydro-demolition. A problem Solving and Decision Making Team (PSDM) team was formed using FENOC personal supported by industry experts from Bechtel and Sargent & Lundy. The team's decision was to "chip back" along the cracked areas and observe the indication. No indications were found on the left side or the bottom after chipping but chipping on the top revealed the crack extended beyond the construction opening.

To facilitate further investigation, the Impulse Response (IR) methodology was employed to investigate the extent of the crack. The IR method consists of generating a stress pulse in a structure by mechanical impact using an instrumented hammer. A geophone is used to monitor the response of the structure to the impulse load. Both the hammer and the geophone are linked to a portable field computer for data acquisition. The record of the hammer force and the geophone response are processed to obtain the mobility of the structure under test. The mobility value is related to the structure's thickness. When a laminar crack exists the mobility reflects the thickness of the upper layer and a higher mobility value is measured. The IR mobility results are relative readings that were used on a comparative basis to identify areas suspected of cracking. Even though the IR method is not a qualified process and IR results were validated using core bores and were judged to be highly reliable in predicting cracking within the range of the instrument. The core bores were inspected in accordance with EN-DP-01512 (Ref. 12) "Shield Building Concrete Examination."

The initial IR testing (Ref. 38, page 461) above the construction opening indicated that the crack appeared to extend approximately 38 feet above the construction opening (to elevation 676). Four core bores were taken above the opening to validate the IR results. These core bores indicated a crack existed near the outer reinforcement mat. A core bore was also used to confirm the absence of the cracking approximately 38 feet above the construction opening. Figure 02 depicts the region of laminar cracking that was identified in the area above the construction opening.

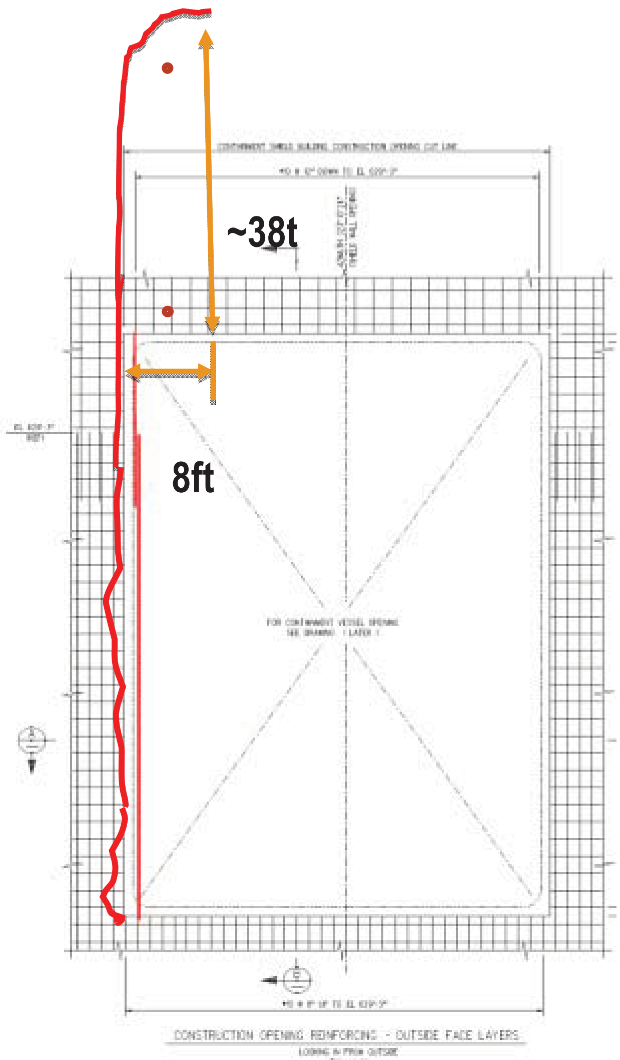

Figure 02 – Extent of Condition at Construction Opening

Although the condition was initially suspected to be caused by the hydro-demolition process adjacent flute shoulders were interrogated using IR testing (Shoulders 1, 13 and 16) to investigate the potential extent of condition in other flute shoulders. These results showed indications of similar conditions in the flute shoulders in both areas tested. Core bores were taken in shoulders 13 and 16 to validate the IR results.

Of particular interest was the four core bores taken in Shoulder 13 to the right of the construction opening. These bores were taken to confirm the relation between the cracking and the lateral reinforcing bar hooks at the shoulder ends. Two bores taken just inside the reinforcing tie bars for the shoulder were not cracked. This in conjunction with the Impulse Response results suggested that the crack was arrested by the presence of the shoulder reinforcing tie bars and was restricted to the shoulder area. This was consistent with the observation that the crack ended when chipping on the left side of the construction opening. Reference 38 page 407 shows the region of cracking that was bounded by the core bores in Shoulder 13.

4.2 Extent of Condition in Flute Shoulders

Additional IR testing was performed on a total of 15 of the 16 flute shoulders (the 16th flute Shoulder as not accessible due to the location of the start-up transformer). The results indicated that the cracking in the shoulders was a generic issue unrelated to the hydro demolition. Core bores were taken on 12 shoulders to confirm the boundaries of the cracking and to characterize the cracks. The boundaries of the cracking were confirmed by nine pairs of core bores performed to validate the IR results on flute shoulders 3, 4, 5, 6, 7, 8, 9, 10 and 12. This demonstrated that the IR results were very accurate at predicting delaminated areas of the flute and shell depths and supported the previous observation that the cracks are confined within the reinforcing steel of the shoulder area, see Figure 04.

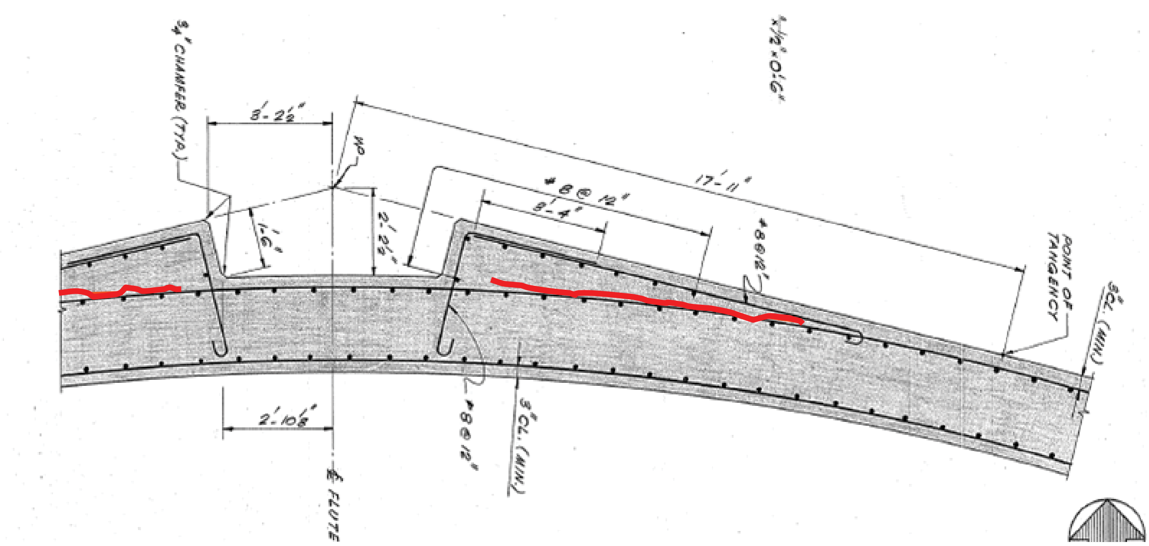

Figure 03 – Extent of Cracking in Flute Shoulder Regions

The core bores were inspected internally using a boroscope in accordance with EN-DP-01512 “Shield Building Concrete Examination” (Ref. 12). These inspections identified and documented the depth and width of the cracks. The crack depths were determined to be near the outer reinforcement steel mat based on a calculated reinforcement steel range. Ground penetrating radar was also used to confirm the calculated reinforcement steel location. The crack width was measured using a boroscope and a crack comparator. All cracks that could be measured were determined to be very tight, less than 0.01 inches. Reference 38 page 404 shows the crack at S12-666.0-4 with a crack comparator.

IR and core bores results taken from scaffolding and man lifts suggested the cracking may have ended at higher elevations. To confirm these results, one flute shoulder (Shoulder 9) was selected to investigate cracking to the top of the Shield Building wall (Approximate Elevation 801). The IR results for this flute indicated cracking exists at various elevations. The indications were confined to the flute shoulder except at the top 20 feet of the Shield Building (approximate Elevations 780 to 801) where cracking is extending into the shell. Reference 38 page 248 shows the results of the IR testing at Shoulder 9. The top region of shoulder 3 was also investigated and indicated a similar condition.

Based on all the investigation of the shoulder regions it was concluded that:

- The cracks are generally confined within the reinforcing steel of the shoulder area.
- Cracking may exist at all elevations in the flute shoulder areas.
- The cracks are very tight, less than 0.01 inches.
- The cracks are near the outer reinforcement mat.

4.3 Extent of Condition Outside of Flute Shoulders

4.3.1 Investigation of Flute Areas

The flute areas are a region approximately 5.5 feet wide between the flute shoulders. The eight flute areas were investigated with IR testing and core bores. IR testing was performed on 4 of 8 flutes. Core bores were taken on 6 of 8 flutes. Both the IR testing and core bores confirmed that laminar cracking was not present in the flute areas. One flute did have a vertical crack in the bore. This was determined to be an isolated condition and was documented separately in Condition Report 2011-04507 (Ref. 6). The absence of laminar cracking in the flute regions provided further evidence that the cracking was constrained by the tie bars of the flute shoulders.

4.3.2 Investigation of Main Shell Areas

Between the flute shoulders there is approximately a 30 foot section of Shield Building cylindrical shell section. IR testing was performed in portions of the shell in 7 of the 8 areas between flute shoulders. The construction opening also provided an opportunity to assess the condition of a region of the main shell. The results of this investigation indicated two regions above the Main Steam Line Room with potential laminar cracking. Four core bores were taken which confirmed cracking in these areas. These core bores were taken to confirm the IR results with regard to the extent of cracking and to identify the crack depth and width. The results indicated the cracking is consistent with previously measured cracks in the flute shoulders. The laminar cracking was near the outer reinforcement steel mat and was very tight.

Investigation was expanded into the Main Steam Line (MSL) Rooms which are below the shoulder areas. Reference 38 pages 672 and 673 shows the results of IR testing in the MSL Room and on the Auxiliary Building roof directly above the MSL room. Testing was performed in regions around the Main Steam line penetration blockouts and in other areas between flutes. The results indicated that cracking was restricted to one side of each MSL penetration blockout. Initially it was suspected that the condition was related to temperature conditions at the MSL penetration. However, an informal analysis on heat transfer from the Main Steam lines did not indicate excessive heat transfer to the Shield Building (Ref. 13). Additionally, the finding that the condition was only on one side of each penetration did not support a temperature related issue. Additional testing was conducted at a similar sized penetration blockout below the MSL rooms in a Fan Equipment room. This testing revealed no potential cracking around the penetration.

Investigation of the main shell areas was also performed in three regions near the top of the Shield Building. Additional data is being acquired now. Based on these results it was concluded that cracking exists in many regions of the top 20 feet of the Shield Building outside the actual flutes. The degree of this cracking in this area appears to vary between different regions of the building. Two core bores were taken to confirm the IR results with regard to the extent of cracking and to identify the crack depth and width. The results indicated the cracking consistent with previously measured cracks in the flute shoulders. The laminar cracking was near the outer reinforcement steel mat and was very tight. IR data would suggest that the spring line area (top 5 feet) is essentially uncracked. Rebar density in the region is too high to obtain a valid core bore. Additional core bores and IR data are being obtained to further characterize the region at the top of the Shield Building wall.

The conclusions of the main shell investigation are as follows:

- The spring line appears to have little or no cracking.
- The flute areas appear to be free of laminar cracks.
- Cracking regions exist at the top 20 feet of the Shield Building wall outside the Shoulder Region. Additional investigation will provide further characterization of these regions.
- Two small regions adjacent to Main Steam Line penetration blockouts are cracked. The extent of cracking in these regions is localized and unique to these particular penetrations.
- Cracks are located near the outer reinforcing mat.
- Cracks are very tight, less than 0.01 inches.

4.3.3 Investigation of Inside Reinforcement Steel Mat

All of the laminar cracking that has been observed and documented is contained in the exterior portion of the Shield Building reinforcing steel mat. The Construction Opening for the reactor head replacement provided a 25 foot wide by 35 foot high perimeter that was inspected for cracking. Visual inspection of the entire perimeter of the construction opening identified the crack location as being associated with the architectural flute shoulder area and near the outer layer of reinforcing steel. No cracking was observed in the inner reinforcing steel mat.

Additionally, the core bores performed in the Shield Building confirmed that the laminar cracking was confined near the outer layer of reinforcing steel. Three deep core bores were taken that reached the interior reinforcing steel mat on Shoulders 16 and Flute 7. These bores showed no cracks in the interior section of the wall. Two other cores with deep bores also revealed no cracks in the deep sections of the Shield Building wall. Two additional deep bores were taken in the upper 20 feet of the wall. Neither of these bores revealed laminar cracks on the interior section of the wall.

Based on these results it was concluded that the cracking is confined to the outer reinforcement mat of the Shield Building and is associated with the cracking observed in the flute shoulder regions. The cracking at the upper level of the Shield Building wall and in the Main Steam Line area is believed to have been caused by the same phenomena that caused the flute shoulder area cracking. This cracking may have extended from the flute shoulder area as a result of discontinuities at these locations. In the case of the Main Steam Line penetrations, the penetrations are in close proximity to the bottom of the flute shoulder area. At the top of the Shield Building there is a discontinuity associated with the interaction from the shallow dome/roof of the Shield Building. It is possible that these discontinuities have played a role in extending the shoulder cracking to these regions. Additionally all indications are adjacent to the architectural shoulders, which present an irregularity on the outside of an otherwise perfect circular shell. Because of this behavior, there is no reason to believe that cracking will occur on the inside face which does not have such irregularities.

Since the crack locations are associated with the exterior portions of the flute shoulder area as confirmed by visual inspection of the construction opening and through core bores. The lack of similar discontinuities on the interior surface of the Shield Building, additional field investigation on the inside portion was not considered warranted.

4.4 Additional Investigation Scope

The region near the top 20 feet of the Shield Building wall was assumed to be cracked in all 360 degrees. This assumption was determined to be overly conservative to evaluate the effectiveness of the hoop reinforcement in the top twenty feet of the building wall. Therefore, additional interrogation of this region has been initiated. This testing will evaluate the following:

- Extent/Existence of cracking in the flute region.
- Extent of cracking in the shell area between the flute shoulders.
- Existence of cracks in the inner reinforcement mat.

4.5 Conclusion of Condition Assessment

The extent of condition was conducted in a systematic manner to identify the extent of the laminar cracking in the Shield Building. IR testing was performed at locations around the building and up to the top of the Shield Building wall. A total of 59 core bores were performed to validate the IR testing and collect data for the root cause team. Overall, the conclusions of this investigation can be summarized as follows:

- Cracking is generic to all flute shoulder regions and can be assumed to be present at all elevation in the shoulders.
- Cracks are confined to the flute shoulder regions with the exception of the top 20 feet of the Shield Building wall and two small regions near the Main Steam Line penetrations.
- Cracking regions exist at the top 20 feet of the Shield Building wall outside the Shoulder Region. Additional investigation will provide characterization of these regions.
- Two small regions adjacent to Main Steam Line penetration blockouts are cracked. The extent of cracking in these regions is localized and unique to these particular penetrations.
- Cracks are located near the outer reinforcing mat.
- Cracks are very tight, less than 0.01 inches.

5.0 Structural Evaluation

5.1 Original Design of the Shield Building

The original calculations documented in USAR Section 3.8.2.2.6 (Ref. 11) followed a very conservative approach to obtain the vertical reinforcement demand for ultimate strength design (refer to calculation VC03-B001-003 (Ref. 14) and VC03-B001-004 (Ref. 15)). This approach results in unrealistically large demands for the vertical reinforcement based on the assumption that concrete remains uncracked under bending moments due to lateral loads, which is inconsistent with the design philosophy of ACI 318-63 (Ref. 18) , ACI 307-69 (Ref. 19) and modern reinforced concrete design codes (e.g., ACI 349.3R-10 (Ref. 20)). Note that concrete cracking shifts the Neutral Axis location; which increases the internal moment arm of the vertical reinforcement, and results in lower demands for the vertical reinforcement. Therefore the demand to capacity ratios reported in USAR Section 3.8.2.2.6 (Ref. 11) for DL + LL + E' + T_A are very conservative and unrealistically large.

There is a significant amount of hoop reinforcement in the top 20 feet (No. 11 @ 6 inches each face) and an additional layer of 10, No. 11 reinforcement in the top 5 feet at the center of the wall section. Review of the original calculation (VC03-B001-001 (Ref. 21) and VC03-B001-008 (Ref. 22)) indicates that the above hoop reinforcement in the top 20 feet was calculated based on conservative methods given the conservative analysis; there is margin in terms of available hoop reinforcement for the design basis loads. This available margin will further reduce the expected induced bond stress in the hoop reinforcement.

5.2 Evaluation of Construction Opening (current design)

The controlling load combinations for the current permanent configuration as identified in Engineering Change Package 2010-0458, (Ref. 23) are in Calculation C-CSS-099.20-046 (Ref. 24). The approach used in this calculation is consistent with ACI 307-69 (Refer to ACI 307-69 supplement (Ref. 19)). This calculation uses a more realistic approach by allowing the concrete in the tension zone to crack which shifts the Neutral Axis and increases the internal moment arm to resist the overturning moment. This results in significant reduction in the reinforcing steel required for the specified load combinations and is consistent with ACI codes.

The reinforcing steel demands calculated in calculation C-CSS-099.20-046 Table 6 identifies the maximum vertical reinforcing steel tensile stresses including thermal effects (i.e., for D + E' + T_A) are about 32% of the yielding stress. Therefore the section has sufficient reserve capacity and the demand in terms of bond stresses is low. Horizontal reinforcement steel stress was not evaluated in this calculation as these stresses were lower and not controlling.

Based on the analysis performed in calculation C-CSS-099.20-046 (Ref. 24), the Shield Building is robustly designed and detailed with significant overcapacity. The maximum stress in the vertical reinforcing steel at the critical section used in the controlling design basis loads is only 32%. This indicates that the actual expected bond stress demand between the reinforcing steel and concrete is significantly less then the allowable values.

5.3 Applicable American Concrete Institute Codes

The issue of cracking was evaluated by (1) a thorough condition assessment using ACI 349.3R-10 as a guide and (2) detailed structural evaluation as discussed in Bechtel's Technical Assessment Report.

ACI 349.3R-10 "Evaluation of Existing Nuclear Safety-Related Concrete Structures" was used because it is appropriate for evaluation of an existing Category I structure. Although, this document does not specifically address laminar cracking, it helps develop a general understanding of the condition and health of the structure. ACI 349.3R-10 was used in part to assess the overall condition of the Shield Building based on visual observation, non destructive testing, invasive and laboratory material testing and structural evaluation. The following provides the purpose and process used as outlined in this document (excerpts from ACI 349.3R-10):

"This report recommends guidelines for the evaluation of existing nuclear safety-related concrete structures. The purpose of this report is to provide the plant owner and engineering staff with an appropriate procedure and background for examining the performance of facility structures and taking appropriate actions based on observed conditions. Methods of examination, including visual inspection and testing techniques, and their recommended applications are cited. Guidance related to acceptance criteria for various forms of degradation is provided."

"For this report, evaluation is defined as an engineering review of an existing concrete nuclear structure with the purpose of determining physical condition and functionality of the structure. This evaluation may include a review of previously accomplished repairs or maintenance, and performing condition surveys, testing, maintenance, and structural analysis."

Observation to date of the Shield Building met the following acceptance criteria given in Section 5.1 of ACI 349.3R-10:

- ✓ Absence of leaching and chemical attack;
- ✓ Absence of abrasion, erosion, and cavitations;
- ✓ Absence of drummy areas (poorly consolidated concrete)
- ✓ Popouts and voids less than 20 mm (3/4 in.) in diameter;
- ✓ Scaling less than 5 mm (3/16 in.) in depth;
- ✓ Spalling less than 10 mm (3/8 in.) in depth and 100 mm (4 in.);
- ✓ Absence of any signs of corrosion in the steel reinforcement;

- ✓ Passive cracks less than 0.4 mm (0.015 in.) in maximum width ; and
- ✓ Absence of excessive deflections, settlements, or other physical movements that can affect structural performance.

The above evaluation indicates that, leaving the structural implications of laminar cracking aside, there are no signs of structural degradation or indications of long-term durability concerns with this structure. It is recognized that ACI 349.3R-10 does not specifically address the laminar cracking; therefore additional evaluation regarding the structural implications of laminar cracking was carried out as discussed in Bechtel's Technical Assessment Report. The above condition assessment together with the conclusions of the Technical Assessment Report (Ref 33) indicate that the Shield Building will perform as intended to meet its design basis load.

All indications are that the cracks observed are passive in nature. However, they were discovered for the first time and there is no historical data at this time to definitively characterize them as "passive". See Section 6.2 for the "Interim Monitoring" plan description.

5.4 Load Transfer between the reinforcing steel and the concrete

The force transfer between the reinforcement steel and concrete is well established in ACI 408.3-01/ACI 408.3R-01, ACI 408R-03 and ACI 408.2R-92 (Ref. 27, 29, 30). Per these references, the transfer of forces from the reinforcing steel to the surrounding concrete occurs by a combination of chemical adhesion, frictional at the interface and mechanical anchorage/bearing of deformed lugs (ribs in the reinforcement steel). These documents indicate that after the initial slip between the concrete and reinforcing steel, most of the force (approximately 80%) is transferred by mechanical bearing between the reinforcing steel ribs and the concrete, see Figure 03. Given the rib dimensions of a No. 10 or a No. 11 bar (0.064 and 0.071 high per ASTM A 615 (Ref 31)), it is reasonable to assume that there is not a total loss of this mechanical bearing between the reinforcing steel and concrete with a tight crack (maximum width 0.01 in).

Bond strength on reinforcement bar in the presence of a known crack is not available. This parameter does not lend itself to calculational methods and is typically established by testing. However, an evaluation can be provided which looks at the actual loading and margins in the structure and compares this to available literature for bonding mechanisms. This evaluation depends primarily on the fact that the loading in the reinforcement mat is subject to very low stresses when compared to allowable values. In some cases, such as the top of the structure, the level of loading shown in the calculations is predominantly driven by environmental effects. If the structure is not capable of handling the imposed loading, visible degradation on the surface would be evident. Because of the calculated low stress levels in the reinforcement under the design basis loads it is expected to have a negligible effect with transfer of force into concrete in the presence of a tight laminar crack.

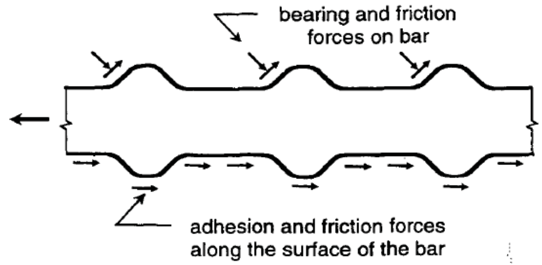

Figure 04 – Reinforcement Load Transfer Mechanism

The development length or lap splice length is a ACI Code value (in inches) used to ensure that the tensile forces from one reinforcing steel bar can be adequately transferred through the concrete to the adjacent reinforcing steel bar. Development and splice lengths per the ACI Code for ultimate strength design were originally developed to achieve strengths beyond the reinforcing steel bar yield. The calculated reinforcing stresses are far below code allowable values (32% for the most limiting load case for vertical reinforcing steel) and therefore full development length or splice length of the reinforcing steel bars is not required for actual load transfer. A reinforcement bar with excessive development overlap which is loaded at less than 50% load presents substantially less challenge to the bonding.

Drawing C-110 Note 7 specifies that all reinforcing steel lap splices shall be staggered which reduces the concern with lap locations concentrating in one region. In addition the lap splices actually provided for the No. 11 hoop reinforcement are 10 ft long (120 inches) from Elevation 780'-0 to 801'-0" as shown on Drawing C-110 and staggered per Note 7. This lap length is more than that required (79 inches) per the Table shown on Drawing C-110 and also more than the lap splice length of 61 inches (1.2*36*bar diameter) required by the original design code (ACI 318-63). Thus the upper portion of the Shield Building has nearly twice (61/120) the required lap length to develop the full ultimate capacity of a No. 11 bar (60 ksi at yield) and significantly more lap length when considering the actual stresses in the reinforcing steel.

The maximum hoop reinforcement stress expected under the worst-case design basis load condition near the top of the Shield Building (Calculation C-CSS-099.20-056 (Ref. 26)) is only 19.8 ksi (1/3 half of that at yield). 13 ksi of this is a result of normal thermal cycles. Therefore, the expected lap length to develop this stress will be only (61 inches * 20 ksi/60 ksi) = 21 inches. In other words, for the maximum stresses we expect under design basis loads, we only need about 21 inches of splice length. This indicates that actual bond stress demand at the splice is expected to be significantly small (21/120 ~18% the design value) because of the available 10 ft long splice. This low stress bond transfer demand has withstood the normal thermal cycles based on the condition assessment of the building.

Based on this review the original design of the Shield Building followed a very conservative approach which resulted in significant overcapacity such that the stresses in the vertical reinforcing steel at the controlling locations are 32% of its Code allowable capacity. The specified lap splice is based on developing full rebar capacity, since the controlling stress is 32% of the allowable, a corresponding reduction in lap splice would also be appropriate. The laminar crack widths are very tight (<0.01 inches) such that the impact on load transfer between the reinforcing steel and concrete will be negligible. Based on the above, it can be concluded that the Shield Building will remain capable of performing its design based function in the current condition.

5.5 Evaluation of Vertical Reinforcing

Since the bonding of the concrete can not be specifically calculated, analysis has been prepared to address the capability of the structure to perform as if the reinforcement were not effective in the area of cracking. Local reduction of bond strength is a concern if the bars are spliced in a region of cracking. Therefore, to address lap splices in the vertical reinforcement two analyses were performed with the outside face vertical reinforcing bars located within 10' strips on either side of each flute, and the 30' strip at the two Main Steam Line penetration locations removed, see Figure 04.

5.5.1 Sectional Analysis

The purpose of calculation C-CSS-099.20-054, (Ref 17) is to evaluate the structural integrity of the Shield Building for all design basis loads, as a Category I structure, after completion of the Reactor Vessel Head Replacement. It is assumed that the outside face vertical reinforcing bars located within 10' strips on either side of each flute, and the 30' strip at the two Main Steam Line penetration locations are not effective (See sketch below). However, since the rebar is in place and the condition assessment is positive this reinforcement is considered to satisfy the code requirements for temperature, shrinkage, ect.

Sectional analyses, based on the principles of strain compatibility of equilibrium, were used after the restoration of the opening. This methodology is consistent with the design basis in the USAR and Design Criteria Manual (Ref 11, 32).

The calculated maximum rebar strains are compared with the rebar yield strain (Table 7 to 10 of the calculation provide the results for different load combination and show sufficient margins of at least 70% for the rebar tensile stress and 30% for concrete compressive stress), and it is demonstrated that the structure will behave within its linear range (no plastic deformation occurs in the reinforcement). Therefore, it is concluded that the structural integrity of the structure will be maintained.

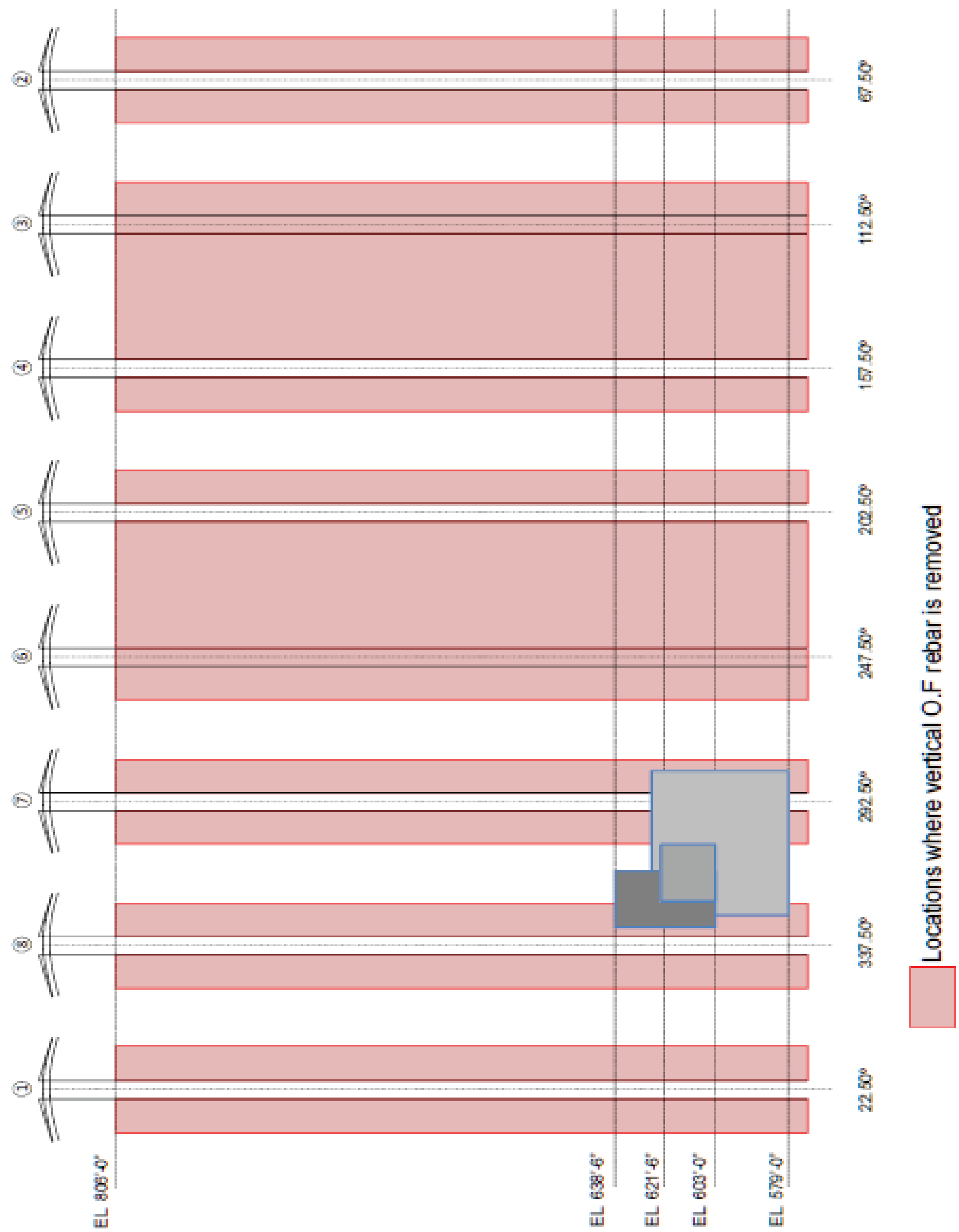

Figure 05 – Location where vertical O.F. rebar is removed – Part 1 of 2

5.5.2 Finite Element Analysis

The purpose of study calculation C-CSS-099.20-053, (Ref 16) is to evaluate the structural integrity of the Shield Building for the interim condition (temporary construction opening in place) for Seismic II/I. This calculation will no longer be required after Shield Building opening is filled.

It was conservatively assumed that the outside face vertical rebars with a 27' wide strip, from bottom to top (EL.565' to EL.809.5'), at each flute shoulder location are not effective. It is also assumed the outside face vertical rebars with 30' wide strip, from bottom to top (EL.565' to EL.809.5'), at two main steam penetration locations are not effective (Figure 05).

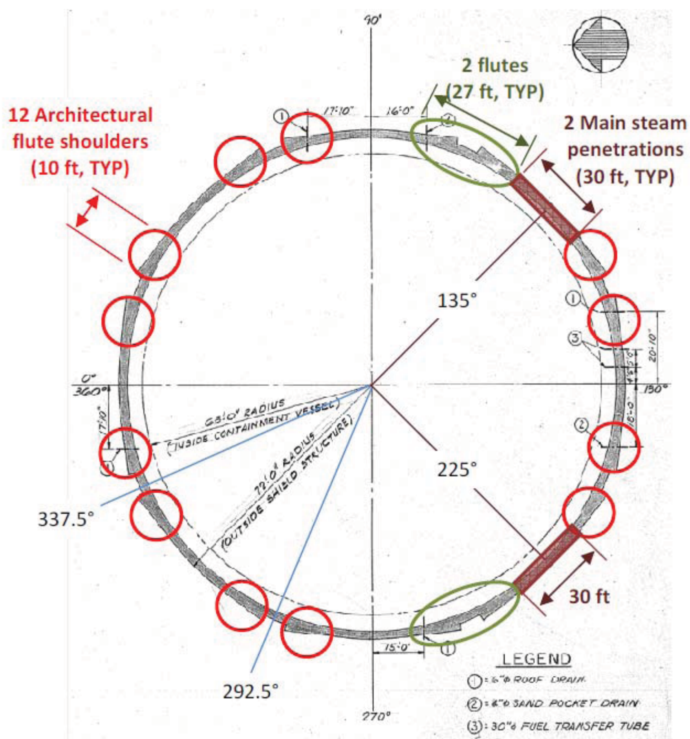

Figure 06 – Location where vertical O.F. rebar is removed – Part 2 of 2

Since this is an interim condition, non-linear finite element analysis was deemed as acceptable and was performed for the controlling load combination D+E'. It was found that plastic behaviors are very limited to no more than 2 elements at the opening corners, while all other reinforcing bars are within the elastic range. The maximum rebar ductility ratio at the corners is less than the acceptable ductility ratio (Refer to Calculation C-CSS-099.20-045, Rev.0, Sect. 7.9, and (Ref.28)).

5.6 Evaluation of Hoop Reinforcement

Calculation C-CSS-099.20-056, (Ref. 26) was initiated to evaluate the stresses in the horizontal hoop reinforcing steel at various elevations of the Shield Building for all applicable design basis load combinations. In particular, four regions were examined in this calculation.

Region 1	Spring Line at elevation 796.5' – 801'
Region 2	780' – 796.5'
Region 3	764.5' – 780'
Region 4	Main Steam line area 584' – 764.5

The maximum hoop reinforcement stress for the worst-case load combination is Region 1 of the Shield Building is 19.8 ksi and in Region 4 the maximum stress is 22.4 ksi. These values are significantly lower than the maximum allowable 60 ksi value.

As discussed previously the effectiveness of the bond for reinforcement can not be explicitly calculated. Local reduction of bond strength is a concern if the bars are spliced in a region of cracking. The reduction in effectiveness of the lap splices in the regions of laminar cracking is addressed in the following three specific regions:

The shoulder regions,
The main steam penetration region and
The top 20 feet of the Shield Building wall.

5.6.1 Shoulder Regions

For regions of the Shield Building below the top twenty feet of the wall each horizontal layer has 20 individual hoop bars that are 30 feet long. Each end of the 30 foot bar has a 79 inch splice. In this region the laminar cracking has been assumed to exist in a maximum eleven foot region of each flute shoulder. This represents approximately 40 percent of the total area of the building. However, based on calculation C-CSS-009.20-056, the maximum hoop reinforcement stress of 23.6 ksi provides a margin of almost three times the maximum allowable value of 60 ksi. Therefore, there is almost three times the amount of hoop reinforcement steel than is required. Since 60 percent of the concrete in the Shield Building in this region is solid and the cracked areas have a maximum width of 11 feet, it would be expected that over half the total splice length of all the reinforcement bars in this region are in good concrete. Given the margin of almost three times the allowable stress, it is reasonable to conclude that the "as-found" laminar cracks will not impair the structural adequacy of the structure.

Each horizontal layer of hoop reinforcement is staggered as specified per Drawing C-0110. This staggering of splices minimizes the potential of having splices located in the same vertically cracked area. Also the Shield Building is curved, which provides additional bearing on the rebar ribs in the interior portion of the horizontal hoop steel. This additional bearing will aid in transferring the forces to the concrete in the cracked areas.

5.6.2 Main Steam Line Penetration Region

The regions identified near the main steam line rooms are small localized areas best described as oval in nature approximately 30 feet wide and 15 feet high surrounded by sound concrete. The horizontal reinforcing in this area is the same configuration that is described in the previous section. This area also has similar stresses as discussed in the shoulder areas where there is almost three times the amount of hoop reinforcement steel than is required. Splices are also staggered per Drawing C-110 in this area.

Since the area is localized, the amount of reinforcing steel provided is approximately three times the required amount, and the splices are staggered, any splices located in this region would be offset by the adjacent splices located in the adjacent sound concrete. Therefore, it is reasonable to conclude that the "as-found" laminar cracks in the Main Steam Line area will not impair the structural adequacy of the structure.

5.6.3 Top Region of Shield Building

The top twenty feet of the Shield Building has additional horizontal reinforcement. At the top region the reinforcing bars are at six inch spaces both inside and outside face of the wall. In addition, the top five feet has an additional layer of #11 rebar at six inch spaces located in the center of the wall.

This area also has indications of cracking in regions outside the shoulder area. Additional investigation is ongoing to better quantify the amount of solid concrete to support bond transfer between the hoop reinforcing steel and the concrete. Currently at least one area has been identified that spans a sufficient horizontal direction that the 30 foot individual hoop bars will not span the region. Additional investigation in progress may identify other similar regions.

Calculation C-CSS-099.20-56 (ref 26) calculates the maximum hoop reinforcement stress for the worst-case design basis load condition near the top of the Shield Building. As presented previously, four regions were examined in this calculation.

Region 1	Spring Line at elevation 796.5' – 801'
Region 2	780' – 796.5'
Region 3	764.5' – 780'
Region 4	Main Steam line area 584' – 764.5

Table 4 below summarizes the maximum reinforcement stresses in various regions identified above. The maximum hoop reinforcement stress for the worst-case design basis load condition near the top of the Shield Building is 19.8 ksi and at the mid-section of the Shield Building is 23.6 ksi. These values are significantly lower than the maximum allowable 60 ksi value.

Table 4: Outside face reinforcement stresses (tension negative)

	Region 1	Region 2	Region 3	Region 4
Load combiantion	f_s [ksi]	f_s [ksi]	f_s [ksi]	f_s [ksi]
$D+T_o$	-13.1	-12.4	-13.2	-12.6
$D+L+E_h-E_v+T_A$	-17.9	-17.4	-21.6	-22.4
$D+L+E_h+E_v+T_A$	-19.8	-18.5	-22.0	-22.4
$D+L+W'+T_o$	-11.9	-13.7	-21.6	-23.6

The stress at the top of the Shield Building is predominately driven by dead load from the dome and normal thermal gradient stresses. Since these stresses are not accident stresses but actual stresses, the Shield Building has already gone through many cycles for this load combination. Had there not been adequate transfer of the stresses between the concrete and the hoop reinforcement in this area, there would be visible surface degradation in the immediate area. This area has been inspected as part of the Impulse Response reading and core bore activities and there has been no signs of surface degradation in this area. Based on the above, it can be concluded that there is adequate confinement of the hoop reinforcement

As discussed previously the effectiveness of the bond for reinforcement can not be explicitly calculated. Local reduction of bond strength is a concern if the bars are spliced in a region of cracking. To account for this, the hoop reinforcement is assumed to be ineffective for any design basis mechanical loads in order to account for reduction of strength as a lower bound. However, all the reinforcement is effective for serviceability, crack control and shielding as it has been for the life of the structure. The calculations are made to demonstrate that applied mechanical loads in these regions are low and well within the capacity of concrete sections without the external hoop reinforcement.

For example, based on Table 4 for Region 2, dead plus normal thermal load results in a stress of 12.4 ksi. We know that region 2 has already experienced many cycles of this stress without causing any visible distress. If we are to assume that, as a lower bound, outer hoop reinforcement is ineffective now for any additional loads, it would mean the following:

1. Additional stress shown under load conditions 2 and 3 of Table 4 will not develop because the reinforcement is ineffective.
2. Additional stress due to mechanical loads (wind) due to load condition 4 is on a cracked section would marginally increase the compressive stress (which are low to begin with) but still well within the compression capacity of concrete. The

section will not reach its nominal capacity which is dictated by crushing of concrete (strain of 0.003).

3. Note that actually there is only a nominal increase in the bond stress at the laps (20 psi at the top of the building for 18.5-12.4 = 6.1 ksi additional stress), which is expected to be carried by the outside reinforcement even if there is a crack along the plane of the bar.

Similar arguments apply for other cracked sections in the shoulders and 2 steam line penetrations where cracking is localized and hoops staggered as discussed in detail below.

5.7 Seismic II/I Evaluation

Calculation C-CSS-099.20-055 (Ref. 34) reviews the seismic adequacy of the shoulder reinforcement in the unlikely event that this architectural feature would act independently from the Shield Building wall. This analysis demonstrates that using a conservative approach, the seismic support of this architectural feature is assured. The analysis concludes that there are several orders of magnitude in the safety margin (about 4.5 per Page 12).

Shear friction and tensile capacity of the #8 rebar is addressed per ACI 318-05 (Ref 35) for worst case scenarios, and is demonstrated as being capable of holding the concrete with sufficient margins.

6.0 Applicability for Continued Operation

6.1 Potential Crack Propagation

As of November 14, 2011, the root cause team formed to investigate the laminar cracking discovered in the Davis-Besse Shield Building has developed a comprehensive list of failure modes and is in the early stages of the support/refute process. The team was formed to continue the investigation started by the problem solving team. Initial efforts by the problem solving team supported by Bechtel Power Corporation and Sargent & Lundy personnel were unable to identify an "obvious" failure mechanism which could explain the cracking discovered. Two prime contractors are supporting the root cause team efforts: Performance Improvement International and MPR Associates.

A total of 15 of the two inch core bores collected by the problem solving team during Impulse Response confirmation were shipped to Photometrics Inc in Huntington Beach, CA. Eight of these core bores contained cracks in the body of the bore and seven were uncracked. All fifteen core bores have been subjected to examination using a Scanning Electron Microscope (SEM) to gain additional forensic evidence as input to the root cause investigation.

The fifteen core bores were examined by the laboratory yielding the following preliminary conclusions:

1. No evidence of micro-cracks between the aggregate and cement
2. No evidence of freeze/thaw micro cracks
3. No evidence of void formation by capillary effects

Rebar samples taken from the Shield Building construction opening were also examined by Photometrics. These rebar samples did not exhibit corrosion damage and were representative of the amount of corrosion one would have expected during the construction process. There was no evidence of extensive corrosion damage that would be expected if the rebar had been exposed to water after the Shield Building had been constructed during the in-service or operational phase of the Shield Building.

A number of larger cracked concrete samples pieces removed from the construction opening were also visually inspected on-site by members of the root cause team. These samples also showed that all laminar cracks had propagated directly through pieces of aggregate indicating that the bond between the aggregate and the concrete was very good and consistent with the core bore samples that have been analyzed to date. These large trans-aggregate laminar cracks indicate large and fast acting fracture forces.

Although all results to date are preliminary, the core bores analyzed to date suggest old cracks. The core bore samples are being examined extensively for micro-cracks. To date, there is no evidence of micro-cracking. Micro-cracks would be an indication of time dependent crack propagation. If no micro-cracks are identified, long term slow progression failure mechanisms will likely be ruled out. In addition, the Shield Building design stresses are low, and do not represent an overload condition or a long term failure mechanism that would themselves cause and propagate cracks. As a result of these early sample evaluations, the current hypothesis is that these laminar cracks are likely old cracks and there is no current evidence of age related failure mechanisms. The evidence at this point would point to a fast-acting failure mechanism, likely environmentally induced that occurred early in the history of the Shield Building.

6.2 Interim Monitoring

It can be concluded that the Shield Building will remain capable of performing its design function in the near and distant future. This is based on the current condition of the Shield Building which meets the design requirements with significant overcapacity and a lack of evidence of an age-related failure mechanism during the on-going root cause investigation. Additionally, the root cause that is in progress is expected to identify actions that may be required for long-term management of the Shield Building. However to verify the continued condition of the Shield Building a corrective action has been created to monitor three areas of interest during the next refueling outage in the Spring of 2012.

The corrective action entered into Condition Report 2011-03346 will require examination of three existing uncracked core bores in the areas of interest that are adjacent to known cracked areas. The purpose of this examination will be to confirm that the cracks in adjacent areas are not propagating. The areas of interest are in a Flute Shoulder area, in the Main Steam Line room areas, and in a Flute. Specifically, the corrective action states:

Perform an examination using procedure EN-DP-01512 on three core cores that have been confirmed to have no cracks but are adjacent to core bores that have confirmed crack identified. These core bores are (Ref. 9):

S9-650.0-9 - located in the Main Steam Line room
S9-666.0-12 - located in the shoulder area
F 4-1-666.0-3 - located in a Flute

The examinations on these three core bores shall be performed during 17RFO.

It can be concluded that the Shield Building will remain capable of performing its design function in the near and distant future. This is based on the current condition of the Shield Building which meets the design requirement with significant overcapacity and lack of evidence of an age-related failure mechanism during the on-going root cause investigation. However to verify the continued condition of the Shield Building a corrective action has been created to monitor three areas of interest during the next refueling outage in 2012.

7.0 Summary

The Davis Besse Shield Building is a very robustly designed building which was erected some 38 years ago. It is primarily a passive structure having only a minor system function of providing a ventilation boundary for the annulus vent system. The building shows no evidence of service degradation in that the external concrete surface is sound, with no evidence of significant cracking, bulging, spalling or settlement damage. It is a simple structure that must stand up under environmental loading (including tornado and seismic loads) and provide a radiation and missile shield function for the containment vessel.

During outage work for the 17th midcycle outage, very tight embedded laminar cracking of an unknown origin was discovered near the outer reinforcement bar mat. A sampling plan was executed to determine the extent of the cracking observed. This cracking was determined to be primarily under architectural shoulders on the outside of the building. It was also noted the laminar cracks extend beyond the shoulders in a very limited area at the bottom of the partial length shoulders over the auxiliary building as well as at the top of the structure. Additional inspections are currently being performed at the top of the building. We believe this sampling method has characterized the extent of the cracking in the structure.

The primary concern based on the observed cracking is the ability of the outside reinforcement bar to perform its intended function. Observations of the cracking placed the crack near the rebar mat. Observations at the construction opening and concrete testing would indicate that the concrete is firmly attached to the rebar mat.

Bond strength literature on reinforcement bar bonding in the presence of a known crack is not available. This parameter is empirically determined in the industry and does not lend itself to precise calculational methods. In light of this fact precise quantification of the load carrying capability of these bars cannot be determined. However, an analysis is provided which looks at the actual loading and margins in the structure design.

This analysis depends primarily on the fact that the stresses that the reinforcement bar is subject to are in all cases very low when compared to the ACI 318-63 allowable. Using this approach it was concluded that the rebar mat of interest is capable of addressing these loads in service. In some cases, such as at the top of the structure, the level of loading shown in the calculations was predominantly driven by environmental loads. If the rebar was not capable of handling the imposed loading, visible external degradation would be evident.

Since the development of the lap splices cannot be specifically calculated, analysis have been prepared to address the capability of the structure to perform without crediting load carrying capability of the rebar in the area of the cracking. Due to the spacing and offsets in the horizontal rebar and the narrow spans of the shoulder cracking areas, it is concluded that there is not a concern with splice stack up resulting in a significant area of reinforcement loss in the horizontal direction below the 780 foot elevation. For the vertical reinforcement steel the structure continues to provide margin to ACI 318-63 allowable.

The analysis developed for the area above elevation 780 recognizes the fact that there are substantial segments with sound concrete. The reinforcement steel in this region is very densely packed with 10 foot lap splices. With this dense configuration, areas have been documented with all horizontal bars spliced in the cracked region. As a result, an analysis of the impact of the design basis loading on the structure is being performed.

Based on this work, we believe the identified cracking does not impact the ability of this structure to perform its intended safety function.

A root cause effort is underway using significant industry expertise. We are hopeful to discover the root cause of this cracking and determine any station or industry learning's for the future.

8.0 References

1. CR 2011-03346, Fractured Concrete Found at 17M Shield Building Opening

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

3. CR 2011-04190, Surface Cracks Identified on Fluted Areas of the Shield Bldg

4. CR 2011-04214, Core Bore Cracking in Shoulder above EDG Building

5. CR 2011-04402, Fractured Concrete found at 17m near MSL Penetrations

6. CR 2011-04507, Misc Indications Identified During Investigation

7. CR 2011-04648, Shield Building IR Indications above El 780 feet

8. DBNPS Drawing C-0110, Revision 6 – "Shield Building Roof Plan Wall Section & Details"

9. DWG C-0111A, Revision 0 - "Shield Building Exterior Developed elevation" Dated 11/3/11

10. Specification No. 7749-C-38, Technical Specifications for the Shield Building, Revision 1

11. Updated Safety Analysis Report (USAR) for Davis-Besse Nuclear Power Station No. 1, Rev. 28

12. Procedure EN-DP-01512, Rev 0, "Shield Building Concrete Examination"

13. Bechtel Calculation 25539-000-M0C-0000-00003, Revision 0 – "Sensitivity Assessment of Concrete Temperature near Main Steam Line in Shield Building Wall"

14. VC03-B001-003, Revision 0 – "Shield Building Wall – Calc. Membrane Stress in Shield Wall"

15. VC03-B001-004, Revision 0 – "Shield Building Wall at Base Floor – Summarized Stresses and Reinforcement Design"

16. Calculation C-CSS-099.20-053, Revision 0 – "Evaluation of Shield Building for the Interim Condition with Outside Vertical Reinforcement Removed at Each Flute Shoulder" – 11/7/11

17. Calculation C-CSS-099.20-054, Revision 0 – “Evaluation of Shield Building for the Permanent Condition with Outside Vertical Reinforcement Removed at each Flute Shoulder” – 10/31/11

18. ACI 318-63, “Building Code Requirements for Reinforced Concrete”

19. ACI 307-69, “Specification for the Design and Construction of Reinforced Concrete Chimneys”

20. ACI-349.3R-10-Evaluation of Existing Nuclear Safety-Related Concrete Structures

21. Calculation VC03-B001-001, Revision 0 – “Shield Building Wall – Calc. Discontinuity at Spring Line (Wall).

22. Calculation VC03-B001-008, Revision 0 – “Shield Building at Spring Line – Determine Discontinuity Stress at Shield Building Spring Line (Independent Check)

23. Engineering Change Package ECP 2010-0458, “SGR-17M -Install Shield Building Construction Opening”

24. Calculation C-CSS-099.20-046, Revision 0 – Evaluation of Shield Building for the Permanent Condition, Dated 08/11/2011

25. Not Used

26. Calculation C-CSS-099.20-056, Revision 0 (Not Yet Issued) “Evaluation of Shield Building Hoop Reinforcement”

27. ACI 408.3-01/ ACI 408.3R-01, “and Development Length of High Relative Rib Area Reinforcing Bars in Tension and Commentary”

28. Calculation C-CSS-099.20-045, Rev.0, “Evaluation of Shield Building for the Construction Opening - SGR-RVCH Replacement”

29. ACI 408R-03, “Bond and Development of Straight Reinforcing Bars in Tension”

30. ACI 408.2R-92, “Report on Bond Under Cyclic Loads”

31. ASTM A 615, “Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement”

32. Davis Besse - Design Criteria Manual

33. Technical Assessment Report (25539-200-C0R-0000-00001-001) – Revision 1 – Dated 11/2/11

34. Calculation C-CSS-099.20-055, Revision 0 – “II/I Evaluation for Architectural Flute Shoulder” – 10/31/11

35. ACI 318-05, “Building Code Requirements for Structural Concrete and Commentary”

36. Review of Technical Assessment Report No: 25539-200-C0R-0000-00001, Prepared by: David Darwin, 10/29/11

37. Shield Building, Davis-Besse Nuclear Plant [Review of Technical Assessment Report No: 25539-200-C0R-0000-00001] Prepared by: Mete Sozen, S.E., 10/28/11

38. “Davis-Besse Shield Building Cracking Investigation and Assessment Report” – Revision 0, Dated 11/3/11

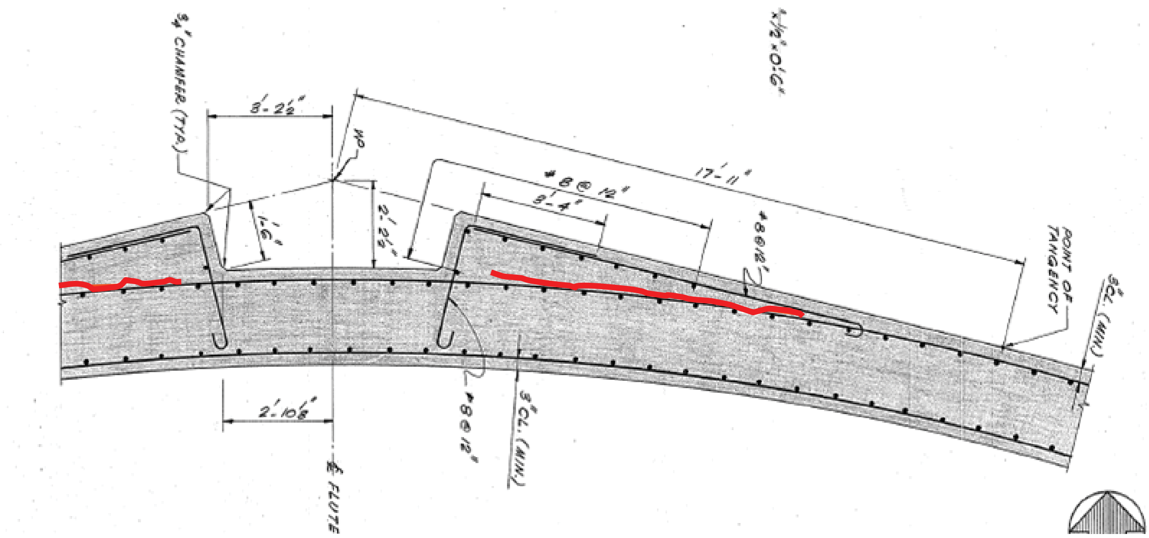

Figure XX – Extent of Cracking in Shoulder Regions