UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
		Docket No. 50-346-LR
First Energy Nuclear Operating Company	)	
(Davis-Besse Nuclear Power Station, Unit 1)		August 16. 2012
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INTERVENORS' FIFTH MOTION TO AMEND AND/OR SUPPLEMENT PROPOSED CONTENTION NO. 5 (SHIELD BUILDING CRACKING)

APPENDIX III: NRC FOIA RESPONSES (B-24)
THROUGH B-26)

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Hernandez, Pete

From: Hernandez, Pete
Sent: Thursday, November 17, 2011 1:55 PM
To: Evans, Michele; Howe, Allen; Lund, Louise
Cc: Mahoney, Michael; Zimmerman, Jacob
Subject: Davis Besse Operability question

Good afternoon Michele,

I understand that the question of Operability vs design basis was posed. And if the Shield Building (SB) issue is in operations space, are qualitative evaluations the extent of review required by the licensee?

To answer that, the distinction between Operability and Functionality needs to be understood. The most clear way I've had it explained is that the determination of Operability is tied to the Tech Specs for the specific plant. If the Tech Specs are met, then it is operable. (An operability determination is usually prompted by degraded conditions, nonconforming conditions or the discovery of an unanalyzed condition.) Functionality is tied to the design bases documented in the FSAR and thereby tied to the Current Licensing Basis.

After clarifying with the region, I've tried to summarize Dave Hills' comments.

The USAR refers to the containment vessel and the surrounding shield building as portions of the "containment system" Besides being required to withstand an earthquake, the shield building serves 3 main functions:

- Limit the release of radioactivity during design basis accidents
- Create a negative pressure with respect to ambient for the Emergency ventilation system to take a suction
- Protect the containment vessel from tornado winds and missiles

The Tech Specs require both the containment and the emergency ventilation system to be operable in Modes 1, 2, 3, and 4. Regardless of whether you chose the term operable or functional with respect to the shield building, the concept is similar with respect to evaluating operability or functionality.

From IMC9900

> "If an SSC described in TSs is determined to be operable even though a degraded or nonconforming condition is present, the SSC is considered "operable but degraded or nonconforming." An SSC that is determined to be operable but degraded or nonconforming is considered to be in compliance with its TS LCO, and the operability determination is the basis for continued operation. The basis for continued operation should be frequently and regularly reviewed until corrective actions are successfully completed."

The licensee's position is that the shield building is operational and conforming. That means it meets all design and code requirements including required safety margins. If they went down the operable but nonconforming route, and if we agreed with the conclusion, they could start up the plant, but we would expect them to have in place a plan to restore conformance at the next reasonable opportunity.

Currently they've given us a qualitative analysis to support their position that the shield building is functional and fully conforming. For NRC to accept and agree, which would mean no additional actions would be necessary to restore conformance, the licensee must provide reasonable assurance to show operability or functionality and provide a logical, supported basis that allows our technical reviewers to reasonably reach the same conclusion. In this case, the qualitative arguments did not provide the logical, supported basis for our technical reviewers to reach the operability conclusion. So we asked if they could provide additional assurance by in some way quantifying their analysis based upon good engineering principles.

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Since we've raised questions, the licensee's concrete/rebar bonding technical consultant, Dr. Darwin (Director of Structural Engineering and Materials Laboratory at the University of Kansas) has informed the licensee that with the assumptions they are making, no credit for the rebar impacted by the cracks is warranted. In light of this, the licensee has started to do more mapping and core bores to better analyze the SB.

Please let me know if I can provide more information.

Thanks,

Pete Hernandez

11/21/11

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DAVIS-BESSE NUCLEAR POWER STATION
CONTAINMENT SHIELD BUILDING ISSUE
11/21/2011

PURPOSE: To inform NRR senior management of situation at Davis-Besse with the Containment Shield Building cracks identified[1], and licensee response.

EXPECTED OUTCOMES: NRR management to understand the status of Davis-Besse's Containment Shield Building (SB).

PROCESS:

Background

While creating an access hole for Reactor Pressure Vessel head replacement, three issues were identified: (#1) Extensive cracking in the shoulder region, (#2) Cracking in the structural region outside the flute shoulder region near the main steam piping penetrations, (#3) Cracking indications via Impact Response (IR) mapping in the cylindrical portion of the building near the top of the building at the interface between the domed roof and the cylindrical wall. Items 2 and 3 are being evaluated separately. IR mapping and core boring continues as the licensee evaluates the top 20' of the building.

Discussion

- On 11/17/11 tech staff from R3 and HQ had a teleconference with the licensee and their experts to discuss the licensee's justification and concerns raised by the professors.
- On 11/18/11 NRC developed and submitted questions for the licensee to answer to provide reasonable assurance that the design basis requirements were met.
- Today, the licensee provided final design basis calculations addressing Items #1 and #2 above. Design basis calculations for Item #3 are pending. The licensee also provided answers to the questions submitted by NRC staff
- Currently in Mode 5 and Mode 4 is scheduled for Wednesday, Nov. 23, 2011.

NRC Questions:

- What quantitative justification is there that the bond strength of the rebar does not need to be derated, or if it does need to be derated, that there was enough margin for safety in place to still meet the capacities in the FSAR? The calculations submitted and the pending calculation are supposed to answer this.
- Has the licensee provided reasonable assurance that the SB will remain capable of performing its design function in the near and distant future (i.e. the condition will not worsen)? Why or why not? They have submitted a plan, but we have raised the question of whether or not we need to approve the plan.

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[1] The issue is not similar to Crystal River. In the case of Davis-Besse, the concrete structure forms the containment building along with the steel liner. The steel containment is a separate structure approximately 5 feet inside the SB (see Picture 2 for a cross-section). There is actually an access area between the containment building and the SB. The SB functions as a portion of the Containment system and is Nuclear Safety-Related, Seismic Category I, and serves 3 main purposes.

- During operations it provides shielding from radiation originating at the reactor vessel and the primary coolant loop
- During operation it provides environmental and tornado missile protection for the containment vessel
- Following a LOCA, the SB serves as a negative pressure boundary for the Emergency Ventilation System.

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Davis-Besse Containment System
Primary Steel Containment and Shield Building
November 17, 2011

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

References:

Davis-Besse USAR Section 1.2.10, "Containment Systems"
Davis-Besse TS 3.6.1, "Containment"
Davis-Besse TS 3.7.12, "Station Emergency Ventilation System"

Description:

As described in the USAR, the "containment . . . consists of two structures: a steel containment vessel and a reinforced concrete shield building, and their associated systems." The steel containment vessel is 2.5 inches thick on the sides and provides a large volume to contain the energy released from a loss-of-coolant accident (and steam line breaks, etc. inside containment). The design internal pressure is 40 psig. This steel structure also must meet 10 CFR Part 50, Appendix J, "Leakage Testing" requirements to limit the release of radionuclides that might exist outside of the reactor system after an accident to a very small percentage of the total volume of the steel vessel.

The shield building is reinforced concrete structure, with a 2.5 feet thick side thickness, that surrounds the steel vessel. There is an approximate distance of 4.5 feet between the shield building internal surface and the steel containment vessel. The shield building "shields" the steel vessel from environmental conditions (rain, snow, wind, etc) including tornado forces and any missiles that might be generated by high wind conditions or turbine failures. The volume between the two structures also provides a path for ventilation (the Emergency Ventilation System) to sweep any radionuclides that might leak from the steel vessel, post-accident, into an Engineered Safety System that can filter (HEPAs and Charcoal) the swept air/leakage before release through a ventilation stack to the environment. This "sweeping and filtering" is utilized to limit exposure to the public, from a hypothetical accident, to below 10 CFR Part 100 guidelines. The shield building also helps reduce the radiation field that might exist, outside of the structures, because of conditions inside the steel containment vessel. The shield building was designed to withstand forces generated by design bases seismic events in addition to forces from temperature changes, etc.

TS 3.6.1 requires that the Containment be operable in Modes 1 through 4. For the primary vessel this requires meeting the leakage limitations and having required valve isolation capability. For the shield building this specification requires periodic visual examination of the surfaces (internal and exterior) of the shield building.

TS 3.7.12 requires that the Emergency Ventilation System, which includes the space between the shield building and the steel vessel, be operable in Modes 1 through 4 (anything greater than 200 degrees F in the reactor coolant system). The Emergency Ventilation System, as part of that specification, must be capable of maintaining a slight negative pressure, relative to existing atmospheric pressure, within the space between the shield building and the steel vessel. This is to ensure that anything leaking, post-accident, from the steel vessel, is swept and filtered prior to release to the environment.

The existing as-found condition of cracking in the concrete of the shield building has raised questions on the ability of the structure to maintain its ability to perform its design functions under conditions that would introduce active forces in the structure (such as a seismic event or potentially rapid changes in environmental conditions).

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1) *How are you categorizing the effectiveness of the reinforcing steel in cracked and un-cracked regions?*

RESPONSE: On the conference call with Drs. Darwin and Sozen both indicated that the capacity of the reinforcement steel after the concrete is cracked (in the 5 to 10 mil range) is still 20 to 30%. This is based on pull tests of straight bars under tensile load. They also indicated that ACI "greatly underestimates the bond strength." In spite of these experienced researchers' position on the ability of the structure to transmit load in the presence of the type cracking seen, we are assuming no credit for the rebar to transmit load through the lap splices in cracked regions in our design basis analysis. This is a very conservative assumption. Dr. Sozen even went so far as to say that he has no concerns with the hoop load failure even if you [discount] the splices all together.

Dr. Darwin summed it up best in his initial comments.

"Overall, I think that the presence of the laminar cracks has the potential to reduce the bond strength of the bars because the cracks are in the same plane as the reinforcing steel. With that said, the local reduction in bond strength is of little concern unless bars are spliced within the crack region. The principal purpose of reinforcing bars is to provide tensile strength and that tensile strength can be provided as long as the bars are anchored to the concrete. If the lap slices are located outside the crack region then, at most, there will be a small discontinuity in strain between the steel and the concrete, but the steel will still serve its intended purpose. Thus, if the splices in the circumferential steel are located outside of the crack region, I agree with and support the conclusion…"[1]

Based on this logic, we need only consider lap splices in the laminar crack region to be ineffective for loads beyond their normal environmental loads. This is based on the observation that they are currently carrying the normal environmental loading (such as seasonal thermal gradient) and have since the structure was constructed.

a) *What is the minimum length of un-cracked concrete required on each side of the rebar in order for the reinforcing steel to carry the design load?*

RESPONSE: Note that maximum stress for the outside hoop reinforcement expected under the worst-case design basis load condition in the cracked region near the top of the Shield Building (Calculation C-CSS-099.20-056) is only 21.7 ksi which is one-third of the ACI 318-63 allowable. Therefore, since bonding load is carried uniformly in a developed reinforcement bar, the expected lap length to develop this stress will be only 21.7/60 x 61 < 24 inches. This indicates that actual bond stress demand at the splice is expected to be significantly small (~ 19 % of design value) because of the available 10 foot long splice.

[1] Dr Darwin's notes from his initial review of the Technical Assessment document

b) *What is the max length of cracked section between uncracked sections of concrete where rebar can be considered to be effective to resist the design loads?*

RESPONSE: The individual horizontal hoop steel is 30 feet long. Since the minimum development length for the max load is 2 feet, the maximum length of cracked region between developed ends would be approximately 26 feet using ACI 318-63 and actual stresses.

c) *What is the basis for the above?*

RESPONSE: The lap splices actually provided for the No. 11 hoop reinforcement in the upper shield building are 10 feet long (120 inches) as shown on Dwg. C-110 and staggered per Note 7. This lap length is more than that required (79 inches) per Table shown on Dwg. 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). If we were to take advantage of the actual concrete strength of at least 6000 psi, the required lap length would further reduce to 56 inches which is less than half of the provided lap length (120 inches). Thus the upper portion of the Shield Building has twice the required lap length. This indicates that the actual bond transfer stresses at the lap will be less than half to develop the full ultimate capacity of a No. 11 bar (60 ksi at yield).

Note that maximum calculated stress for the outside hoop reinforcement expected under the worst-case design basis (ultimate strength) load condition in the cracked region near the top of the Shield Building (Calculation C-CSS-099.20-056) is only 21.7 ksi. Therefore, the expected lap length to develop this stress will be only 21.7/60 x 61 inches < 24 inches. ACI 318-63 specifies a lap splice of length 61 inches which is approximately one half of what was provided. Therefore the required lap splice per ACI 318-63 for our actual loads would be approximately 24 inches.

2a) *In order to demonstrate the Shield Build has sufficient areas of uncracked concrete to support the shield building reinforcing analysis/ load, what percentage of the Shield Building has to be uncracked?*

2b) *What is the basis for justification for this percentage?*

2c) *What are you doing with your mapping to support the design analysis?*

RESPONSE: As was established in response to question one, the critical aspect to reinforcement steel development is the lap splice region. The structure of the Shield Building has exhibited the ability to carry the load imposed by the environmental conditions since it was constructed. These environmental loads are a substantial percentage of the peak load combination for various sections of the structure. Although Drs. Darwin and Sozen both indicated that the capacity of reinforcement steel after it is cracked is still in the range of 20 to 30 percent, since it is not quantifiable based on

current industry knowledge, we conservatively assume it can carry no load under design basis conditions.

Lap splices entirely within the crack zone are conservatively assumed to give way and fail to transfer load. In a large concrete structure the reinforcement steel and concrete act in a membrane fashion. If a local lap splice is ineffective the load will transfer to the adjacent load carrying members. Local structural failures would only exist if a large number of lap splices were to line up in the same crack area. The horizontal reinforcement bars in the shield building were well staggered to preclude this very issue.

Since the maximum stress in the bars under consideration is approximately a third of the allowable, two ineffective splices adjacent to each other could transfer their load to the next available bar and still maintain the allowable. Using this approach suggested by Drs. Darwin and Sozen, it would take greater than four sequential bars not developed to place the adjacent reinforcement (top and bottom) near allowable values. It is important to note that not all the redistributed load goes to only the next available bar. This load acting through the membrane of concrete is distributed to several adjacent bars. This is also a local affect. Once the bars are developed in good concrete, the next laminar crack region behaves in an independent fashion and the same behavior would apply.

Since the reinforcement steel development specified staggered bar splices and the reinforcement steel is lightly loaded, Dr. Darwin suggested that the development could be evaluated on a percentage basis. That is, if the loading in the section is one third of the allowable, then at least one third of the section must contain solid (uncracked) regions to fully utilize the reinforcement steel.

Mapping efforts are being used to characterize the extent of cracking. Impulse Response (IR) signals are used to identify areas of suspected cracking. Core bores are used to confirm the IR readings. Any readings beyond a threshold which has been confirmed by core bores is assumed to be cracked. Conservative assumptions have been made to limit the extremely difficult data collection efforts. This information is being used as input to the analysis to address the shield building cracking issue.

3) *How will your extent of Shield Building mapping demonstrate that you have sufficient uncracked concrete if the entire area is not mapped? If the entire shield building is not mapped what is the justification to extrapolate to other areas of the building?*

RESPONSE: Dr. Darwin addressed this in his comments Thursday night. In order for the structure to be considered sound we should establish a percentage of sound concrete greater than the percentage load calculated in the reinforcement steel. Specifically he stated that we needed to estimate the percent of cover and that there was no need to inspect every square inch of concrete.

Impulse Response testing in conjunction with core bores is being used to characterize the extent of cracking on the shield building surface. As was discussed in the condition evaluation paper, based on the pattern of the indications, sufficient data has been acquired that demonstrates the cracking is limited to the shoulder regions, the small areas at the end of the shoulders near the blockouts for the Main Steam Lines, and near the spring line of the building. There is no evidence to support that the cracking is present generally in the remainder of the shield building shell regions. Additional exploration is being performed to determine the extent of the cracking near the spring line of the building. Accessible areas are being IR tested and confirmed with core bores. Based on the consistency of the data in the regions mapped some areas are assumed to exhibit similar behavior.

4) *Confirm that both vertical and horizontal rebar if located in a crack region are not considered in the strength evaluation*

RESPONSE: The vertical and horizontal reinforcing steel is expected to be at least partially effective in the cracked regions and fully effective in uncracked regions as only minimal bonding along the reinforcing length will achieve adequate strain compatibility around the circumference. In the vertical strips under the shoulders the IR data, in conjunction with core bores, would indicate several regions of uncracked concrete. Although the extra long lap splices for hoop reinforcement are expected to be able to develop some bond stress, the amount can not be quantified at this time.

As such, vertical reinforcement in cracked regions is conservatively considered ineffective in each flute shoulder and in two vertical strips in the main steam penetration areas. This impacts the design load conditions that control at the bottom of the Shield Building. This conservatively evaluates the vertical reinforcement and is based on sectional analysis that does not include the beneficial confinement of hoop reinforcement.

For the hoop reinforcement, the lap splices of hoop reinforcement are considered ineffective for developing any additional load beyond what it has already experienced during normal thermal cycles. For the cracked shoulder areas and steam line penetration areas, conservatively half of the outside face hoop reinforcement is considered ineffective, since each horizontal layer of hoop reinforcement is staggered as specified per Drawing C-110. The probability of laps falling in the cracked regions along with staggering of splices and limited area of crack indications eliminates the potential of having all or most splices located in the same vertically cracked area. The hoop reinforcement is continuous and will develop full capacity outside the cracked region. Note that the vertical and hoop reinforcement is actually present and sufficiently bonded and will provide the necessary serviceability requirements such as crack control as it has under normal operating conditions since the structure was built.

Calculation C-CSS-099.20-054 addresses the conservative effect of cracking on vertical reinforcement using a sectional analysis. Since the effect of cracking on hoop

reinforcement lap splices cannot be captured in sectional calculation, elastic finite element analysis is used to address this in Calculation C-CSS-099.20-056. These two calculations together conservatively address the effect of cracking on both vertical and horizontal reinforcement for the most critical design basis load conditions at various sections along the height of the Shield Building.

5) *Reference the Summary report, page 24, third paragraph. Ensure that the required rebar bond strength will carry the entire design load (18.5 ksi) plus adjacent load from adjacent rebar in cracked area. (Why are only accident loads applied to the upper region?)*

RESPONSE: Calculation C-CSS-099.20-056, Evaluation of Shield Building Hoop Reinforcement with Observed Cracking, provides a bounding analysis of the condition of the top 20 feet of the shield building that assumes hoop reinforcement is ineffective for developing load beyond what is experienced during normal environmental cycles. Table 6(a) and 6(b) of this calculation indicate that under normal dead plus thermal load, the outside reinforcement in the cracked region at the top (Region 2) has already experienced/resisted a tensile stress of 12.4 ksi. The Table also shows that a maximum stress of 21.7 ksi is expected in this reinforcement under combined dead, seismic and thermal load and 13.7 ksi for dead, wind and normal thermal load. Since we assume that outside reinforcement is to be treated ineffective in carrying any additional stress beyond 12.4 ksi, under accident thermal loads that may cause stresses in excess of what the rebar can carry (assumed to be 12.4 ksi), the reinforcement is assumed to detach itself from the outer section of the shell. Because there is no restraint provided by the reinforcement, the accident thermal gradient will tend to self relieve, albeit trying to cause an increase in the crack width until the section finds a new balance. Because of the rigidity of the shell and compression on the inside face due to a moment gradient, it is impossible to develop a through thickness crack in a localized cracked region. This new section balance will be largely affected by the amount of reinforcement that is bonded with concrete in the surrounding areas (above and below the cracked region). In other words, as the section in the cracked region wants to crack, it cannot do that locally and will thus result in redistribution of forces around the cracked region.

As a practical matter, our recent data collection efforts have resulted in identifying significant uncracked sections at the top of the building. We anticipate there will be a percentage of sound concrete greater than the percentage load calculated in the reinforcement steel. This would be consistent with the conditions Dr. Darwin considered adequate for the structure to be considered sound. However, one region has been identified which is longer than the reinforcement steel. The following is offered to support the soundness of using percentages in development even in significantly cracked areas.

For a 30 foot wide by 12 foot high section of cracked concrete in Region 2, there is significant amount of hoop reinforcement above (in spring line area, 3 layers of #11 @ 6 inches) and below (#11 @ 12 inches) that is well bonded with good concrete and will

play a significant part in taking on the additional load redistributed from the cracked regions. For a 12 foot high section, the required reinforcement assuming 21.7 ksi hoop stress requirement per bar is 520 ksi (6 #11 bars). Generally speaking, 4 bars will be provided by the margin available in the reinforcement above (where capacity is 3 layers of #11 @ 6 inches). The capacity equivalent to 2 remaining #11 bars will be resisted by reinforcement below the cracking region (where capacity is #11 @ 12 inches). This is reasonable since the region above is stiffer due to larger reinforcement ratio and will attract more loads in force redistribution.

Therefore, there is significant margin in hoop reinforcement above and below this cracked region (maximum stresses are only one third of ACI 318-63 allowable) to carry this additional load to keep the cracks tight and provide the required shielding and allow the shield building to perform its intended safety function.

SUMMARY:

The cracking observed has been very tight and ranges in depth from the outer cover region to various depths in the outer reinforcement steel layer. Therefore, all the cracking is not in the same plane. Conservative assumptions have been made as to the extent of cracking to limit the complex data taking activity. Industry experts in the field of reinforcement steel bonding have indicated that considerable bonding should exist even if the cracking were all in the same plane. The building steel detail is far in excess of what was required for the loads imposed. The building is well detailed with generous development and splice lengths for the reinforcement. Reinforced concrete structures tend to redistribute applied loads and are not sensitive to local weaknesses in any single area of reinforcement.

The Shield Building is a robust concrete structure (wall) serving as a barrier from external hazards to the containment and a ventilation barrier for the annulus vent system. It must withstand environmental loads which are mild compared to its capacity to withstand these loads. This is evidenced by the conservative analyses which remove significant sections of load transmission capabilities (reinforcement within shoulders, near steam line penetrations and at the top of the building) and still demonstrate adequate load carrying capacities. These capabilities are known to the engineers who work in this field but are not credited explicitly due to our inability to quantify the loads. Even under these extreme assumptions, the analyses indicate the structure is capable of performing its intended safety functions.

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Sakai, Stacie

From:	Sheikh, Abdul *nrr*
Sent:	Tuesday, November 22, 2011 10:51 AM
To:	Sanchez Santiago, Elba
Cc:	Hoang, Dan; Manoly, Kamal; Sakai, Stacie
Subject:	Questions for the Conference Call

There are several documents (summary report, new calculation 056) that have different assumptions and approaches. I did not have enough time to review the calculation (196 pages). However, the basic questions are as follows:

1. What is the actual condition of the concrete 20 feet below the spring line based on field verification.
2. Calculation C-CSS-089.20-056, page 5 states in the assumption section that, "because the bond strength of reinforcement with laminar cracking next to it cannot be quantified, outside face hoop reinforcement in these regions is treated as ineffective --- for ultimate strength calculations." If this assumption is correct only 3-4 inches of the concrete on the inside face can be used in the structural analysis. In the response to the questions, the applicant stated that, " Since we assume that outside reinforcement is to be treated ineffective in carrying any additional stress beyond 12.4 ksi, under accident thermal loads that may cause stresses in excess of what the rebar can carry (assumed 12.4 ksi), the reinforcement is assumed to detach itself from the outer section of the shell." These statements seems to be contradictory. In addition, I am concerned that the concrete will fail in this region due to bending in this region even under small loads.
3. Lap splice issue. ACI 318-63, section 805 (b) states that, "---however, length of lap for deformed bars shall be not less than 24, 30, and 36 bar diameters for specified yield strength of 40,000, 50,000, and 60,000 psi, respectively."
4. At places in the licensee documents, it is stated that due to staggered lap rebar splices, only 50 percent of the rebars are considered effective. If this is the assumption, stress used for lap splice calculation should account for 100 percent increase in the stress.
5. The licensee justification for ignoring the dead (DL) and normal thermal (To) in calculation of rebars splice does not appear to be justified. The stresses due to dead load and thermal loads will be locked in the rebars and cannot be ignored.
6. The licensee considers the allowable stress in the rebar to be 60 ksi and ignores a phi factor (0.9) in his evaluation for lap splice. In addition, the licensee has not accounted for any additional uncertainty due the field conditions.
7. Licensee response to question 1 states, "On a conference call with Drs Darwin and Sozen both indicated that the capacity of the reinforcement steel after the concrete is cracked (in the 5-10 mil range) is still 20 to 30%. This is based on pull tests of straight bars under tensile loads." I am not aware of any pull tests carried out with a crack in the plane of the rebar. Can the licensee provide any documentation for this statement.
8. The licensee is using numerous assumptions in his summary report and calculations that are not described in the UFSAR and ACI 318-63, and still calls it a design basis calculation. Can the licensee provide justification for this approach.

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