Good afternoon Michele,

I understand that the question of Operability vs design basis was posed and that if this issue was 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.

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 decided to not enter into an Operable but Degraded or Nonconforming determination and that the cracking issue is a design basis question hence functionality.

Speculating: The cracks in the building qualify as an unanalyzed condition so for the licensee to Operate with a degraded or nonconforming condition, they would have to develop a plan to fix the issue through their CA process. However, the licensee has stated that the SB is Operable as is, so there is nothing to fix. This still leaves the issue of the cracks unresolved so they are trying to prove that the cracks do not affect the functionality of the building. This led them to the design basis evaluations.
Elba, here are the questions I had about the report from Davis Besse. The calculations were a bit out of my range but, I had questions about their general methods. The Tech folk should have their questions over to you this morning also.

Thanks,

Pete

Questions about Davis Besse Shield Building Report

Objective or Purpose (paragraph 3): In this calculation the structural integrity of the SB is evaluated considering the presence of an interfacial/circumferential crack between the SB structural concrete shell (i.e., the 30" thick reinforced concrete SB) and each architectural flute shoulder (16 flute shoulders in total), as described in Attachment B.

This description makes me think that they are looking at a single crack going in a circle. From what I understood the crack is pervasive along the entire surface, spidering in all directions, similar to a pane of tempered glass breaking. The description in Attachment B addresses only the crack at the opening and assumes that the crack is right along the rebar line. The core bores have shown that the cracks are at different depths so this doesn't seem to capture the current situation. Throughout the calculation, the word Crack, singular, is used. They also mention that the extent of the crack is only 10'-12'. This seems to greatly downplay the issue.

Scope of Calculation/Revision (bullet 4): Maximum concrete crack width under flexure is calculated and compared with the allowable value (Section 7.5). Note that this maximum crack width evaluation only applies to the structural concrete (i.e., the 30" thick reinforced concrete SB shell). In particular, the width of any cracks in the 16 nonstructural architectural flute shoulders is not addressed.

At this point core bores of only the shoulders have been taken. So the only crack widths we are aware of are those in the shoulders, which are not being addressed. How can an analysis be done on the structurally credited concrete if no data from that area, in the form of core bores, has been taken? Shouldn't the structural integrity of the shoulders be calculated as well?

Section 3.0 Methodology (last sentence): Thus, this calculation focuses on the structural integrity of the reinforced concrete within and around the RCVH/SGs construction opening, once it is restored.

This seems to say that they are just doing calculations for the new concrete that is and ignores the rest of the building altogether. Is that right?

Section 3.1 Construction sequence (page 6, second paragraph): However, the vertical reinforcement next to each flute (i.e., in a vertical strip approximately 10 ft wide) is conservatively ignored for evaluating the structural integrity of the SB under mechanical loads.

This says to me, that they are ignoring the shoulders, if they are ignoring all that concrete, it seems to be the opposite of conservative for evaluating the mechanical loads.

C-CSS-099.20.055

Objective or Purpose: The purpose of this calculation is to demonstrate that during a seismic event, with the development of the crack in the architectural flute shoulder, the capacity of rebar(s) can still provide adequate anchorage thus prevent cracked concrete piece from falling, and therefore Seismic II/I condition can be maintained.

I think the greater concern is will the SB stay standing and not whether or not the decorative concrete will fall off. Because the licensee has not performed core bores to see if there is cracking in the credited concrete, do they have a basis to say that the structural concrete will maintain a Seismic II/I condition?
This use of singular terminology also discounts this calculation because it seems that they are looking at only 1 crack and 1 shoulder or 1 flute. Because cracks have been found through multiple core bores, shouldn't the appropriate calculations account for the combined effects of cracks in all the shoulders and not just one by the opening and not just individually?

Section 6.2 (page 7): Based on impulse Response testing, the actual crack length is 10 to 12 feet long. From what I understand, IR mapping is only an indicator, but must be validated by core bores. Does basing all the calculations on a length of a 12 foot crack discount the calculations altogether, because we have indications of cracks at distances greater than 12 feet. This also seems to assume that there is only 1 crack and not many as the core bores seem to prove. Isn't IR mapping only useful at a limited depth too, so that using it to evaluate a 48" thick piece of concrete is not realistic?
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.