UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of NEXTERA ENERGY SEABROOK, LLC (Seabrook Station, Unit 1)	Docket No. 50-443-LA-2 ASLBP No. 17-953-02-LA-BD01

Hearing Exhibit

Exhibit Number: INT032

Exhibit Title: Rebuttal Testimony of Victor E. Saouma, Ph.D Regarding Scientific Evaluation of NextEra's Aging Management Program for Alkali-Silica Reaction at the Seabrook Nuclear Power Plant (Non-Proprietary Version)

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In the Matter of	)	
NextEra Energy Seabrook, LLC	)	Docket No. 50-443
(Seabrook Station, Unit 1)	)	

PRE-FILED REBUTTAL TESTIMONY OF VICTOR E. SAOUMA, PH.D
REGARDING SCIENTIFIC EVALUATION OF NEXTERA'S
AGING MANAGEMENT PROGRAM FOR ALKALI-SILICA REACTION
AT THE SEABROOK NUCLEAR POWER PLANT

SUBMITTED ON BEHALF OF C-10 RESEARCH AND EDUCATION FUND

August 23, 2019

REDACTED VERSION SEPTEMBER 11, 2019

AVAILABLE TO PUBLIC

List of Figures

EXHIBIT INT028

~~CONTAINS PROPRIETARY INFORMATION – SUBJECT TO PROTECTIVE ORDER~~

A Introduction

A.1 Please state your name and employment.

My name is Victor E. Saouma. I am Professor of Civil Engineering at the University of Colorado in Boulder. I am also the Managing Partner of XElastica, LLC, a consulting firm. In addition, I am *Professeur des Universités* in France.

A.2 Do you consider yourself qualified to fully respond to NextEra's and the NRC Staff's testimony?

Yes. As earlier indicated, I consider myself an expert in ASR, finite element analysis, fracture mechanics, computational and experimental mechanic. I have also taught reinforced concrete design, advanced reinforced concrete, finite element, and fracture mechanics.

I have nearly 15 years of continuous research on ASR, 11 major research projects, one book, 5 major reports, 3 short courses, 11 published peer reviewed papers, 5 more submitted, all related to ASR. I was a key contributor to EPRI's report on ASR, Modeling Existing Concrete Containment Structures; Lessons Learned. 3002007777 (2017). For the past four years, I have chaired an International committee (through RILEM (French acronym of International Meeting of Laboratories and Experts of Materials, Construction Systems and Structures)), addressing the diagnosis and prognosis of structures affected by ASR. And I serve as editor of a RILEM report with over 450 pages, and 30 contributors among the top researchers on the related topic of ASR. I have also been President of the International Association of Fracture Mechanics for Concrete and Concrete Structures (and hence am quite familiar with issues pertaining to cracking in concrete). I have advised the Tokyo Electric Power Company (TEPCO) on nonlinear dynamic analysis of large arch dams subjected to strong seismic excitation, conducted shear tests for them (and for EPRI), and consulted for a massive reinforced concrete structure suffering from ASR.

I am the past President (and Fellow) of the IA-FraMCoS, International Association of Fracture Mechanics for Concrete and Concrete Structures

In addition to my training and experience as a scientist, I am also a trained and experienced civil engineer. Most of my research funding has been from sponsors seeking advanced scientific based solutions to practical engineering problems. I have taught linear and nonlinear structural analyses reinforced and advanced reinforced concrete design. Therefore, I am familiar with and able to evaluate NextEra's engineering-based approach to the problem at ASR at Seabrook.

In studying ASR over many decades, I have found that ASR is an extraordinarily complex and nefarious reaction. While it has been known since the 1940's, only recently have we witnessed an emergence of structures suffering from this problem (as it may take many years to manifest itself). As a result, ASR has attracted the attention of researchers from many disciplines:

chemists, mineralogists, geologists, material scientists, mechanicians, experimentalists, and yes structural engineers. Not a single one of those disciplines can provide a definite answer to questions posed by ASR. However, those who have taken a comprehensive view to the problem are best positioned to opinionate. By virtue of the diversity of my research and publications, and my leadership in an international committee addressing ASR with some of the best researchers in the world, I have acquired a global understanding of the problem that position me to opine with confidence on the adequacy of the work done at Seabrook.

A.3 Are you a Professional Engineer (PE)?

No, I am not a licensed PE. I found no necessity to obtain a PE as my consulting contracts for commercial engineering projects (TEPCO, Tropicana casino parking (for Weidlinger & Assoc.), Gilboa dam, Crystal River nuclear power plant, to name a few) generally require knowledge and expertise far beyond those of a PE.

A.4 Please identify this document.

This is my written pre-filed rebuttal testimony regarding my scientific evaluation of NextEra's Aging Management Program for Alkali-Silica Reaction (ASR) at the Seabrook nuclear power plant. My written pre-filed rebuttal testimony is submitted in two versions: **EXHIBIT INT028** is my complete testimony**,** and includes some proprietary information. I am also submitting **EXHIBIT INT029,** which contains the introductory section and a summary of my conclusions. I also plan to submit a redacted version of **ExhibitINT028** as soon as possible.

A.5 What is the purpose of your Rebuttal Testimony?

The purpose of my Rebuttal Testimony is to respond to criticisms of my Opening Testimony by NextEra and the NRC Staff, and to confirm my continuing professional opinion that the large-scale test program (LSTP), undertaken for NextEra at the Ferguson Structural Engineering Laboratory (FSEL) of the University of Texas, has yielded data that are not representative of the progression of ASR at Seabrook; and that as a result, the proposed monitoring, acceptance criteria, and inspection intervals are not adequate.

A.6 Your pre-filed Opening Testimony had about 25 questions and answers. In response, you have received a total of more than 400 questions and answers from twelve witnesses (53 Q/A from NRC Staff, 125 Q/A from SGH, and 236 Q/A from MPR). Are you going to reply to each one of those 400-plus Q/A?

While I have reviewed all of the testimony by NextEra's and the NRC Staff's witnesses, it would not be possible for me to respond in this document to all of their statements. I will focus on the most relevant and significant statements.

A.7 What documents have you reviewed in preparing your rebuttal testimony?

I have reviewed the position statements filed by NextEra and the NRC Staff, and the testimony of their expert witnesses. The documents I reviewed consists of the following:

- Testimony of NextEra Witnesses Michael Collins, John Simons, Christopher Bagley, Oguzhan Bayrak, and Edward Carley (July 24, 2019) (Exhibit NER001) (MPR Testimony);
- Testimony of NextEra Witnesses Said Bolourchi, Glenn Bell, and Matthew Sherman (July 24, 2019) (Exhibit NER004) (SGH Testimony);
- NRC Staff Testimony of Angela Buford, Bryce Lehman, and George Thomas (July 24, 2019) (Exhibit NRC001) (NRC Staff Testimony);
- NRC Staff Testimony of Jacob Phillip (July 24, 2019) (Exhibit NRC005) (Phillip Testimony);
- NextEra Energy Seabrook LLC's Statement of Position (July 24, 2019) (NextEra SOP); and
- NRC Staff Initial Written Statement of Position (July 24, 2019) (NRC Staff SOP).

A.8 According to NextEra, "The LAR is based on sound science and well-established engineering principles and is fully compliant with applicable codes and regulations." NextEra SOP at 2. Do you agree that both scientific and engineering principles were well-applied here?

No. Everything begins with science. When science is well understood, we can translate scientific principles to engineering and eventually write codes. With ASR, one must begin with an adequate scientific understanding in order to verify the adequacy or appropriateness of the engineering principles to apply.

All engineering approaches to ASR should be supported by accurate assumptions. In this case, NextEra applied engineering models without first ensuring that the underlying assumptions were scientifically sound. Therefore, it is now necessary to go back to science and make sure that ASR is well understood according to sound scientific principles. Only then can an adequate engineering approach, i.e., the development of codes and acceptance criteria, be devised or

undertaken. I would also add that any engineering codes that are applied to the problem of ASR must be up-to-date and suitable to the problem.

In this case, NextEra took an engineering method that was specifically created for the initial design of the plant and "tweaked" it for purposes of addressing ASR at the long-operating Seabrook reactor. The application of an outdated design engineering code to a current operational condition yielded an analysis that was inadequate for either diagnosing the ASR problem at Seabrook or conceiving an effective monitoring plan.

A.9 Can you explain in simple terms, possibly through an analogy how you view the differences in approach?

I see a very strong analogy between the Seabrook Containment Enclosure Building (CEB) and a patient suffering from cancer. Indeed, ASR has often been casually referred to as "cancer of the concrete." In the 21st century, when a patient has cancer, the general practitioner refers her/him to a specialist (oncologist). The specialist in turn, performs state of the art laboratory tests, then a diagnosis is first established (extent of the cancer), a prognosis is given of the likely course of the disease, and a treatment plan is established. The treatment plan will include additional laboratory tests to monitor the cancer and determine if the cancer is in remission, spreading, or metastasized. In the worst-case scenario, the patient will be told his or her chances of survival.

In the case of the Seabrook ASR, NextEra is pursuing methods more appropriate to the 19th or 20th Century. There is no written record that NextEra consulted an ASR specialist for the diagnosis or prognosis of ASR. An outdated and flawed tool (a 1971 design code) and primitive surficial observations (comparable to auscultation by stethoscope in a medical context) were used to make a simplistic diagnosis and questionable prognosis of a "slow evolving reaction." The treatment plan included a monitoring program that was based on the unjustifiably optimistic conclusions reached during the flawed steps of diagnosis and prognosis. Finally, the whole problem was left entirely in the hands of "general practitioners" (*i.e.,* engineers), without the assistance of any specialists. Despite their expertise to deal with common nuclear plant maladies, they lack the specialized expertise to make a sophisticated diagnosis or prognosis, to create an adequately informed treatment plan for monitoring the complex problem of ASR, or to analyze and respond appropriately to the monitoring results.

A.10 Can you name examples where so-called scientific approach was followed in lieu of an engineering one for CEB?

Yes, the analysis of the Gentilly-2 (G-2) nuclear plant by Gocevski (Exhibit NER038) at Hydro-Quebec (HQ) is a perfect example. It has been referenced by NextEra, yet the Seabrook analysis is much less sophisticated than the one carried by their Canadian counterpart.

In addition, there are many other examples for dams and even bridges. For dams, an example of modern safety assessment can be found in the report by the Swiss Committee on dams: Swiss

Committee on Dams, *Concrete Swelling of Dams in Switzerland* (2017) which took a very comprehensive and scientific approach to fully understand the impact of ASR on dams. (Note that all the authors are either practicing engineers or employees of utility companies).

A.11 Can you be more specific and contrast what HQ did for G-2 that was not performed by NextEra for Seabrook?

Hydro-Quebec performed a very detailed safety assessment of Gentilly-2 which suffered from ASR. Like NextEra, HQ was seeking to demonstrate compliance with an industry code (in that case CSA Standard N-278). The approach followed by Gocevski in Exhibit NER038 is indeed very much in line with what I have been advocating in terms of rigor and reliability. It is to be contrasted with the very simplistic approach taken by NextEra and approved by the NRC.

Some of the key differences between HQ's evaluation of ASR at Gentilly and NextEra's evaluation of ASR at Seabrook are:

- HQ used a much more sophisticated approach than NextEra in considering humidity. Humidity distribution was considered, including the impact of internal relative humidity. *Id.* at 15. These humidity-factors were found to have a significant impact on the long-term expansion rate and the accumulated total expansion of ASR-affected concrete structures. *NextEra, in contrast, ignored the impact of internal relative humidity in both its CI measurements and finite element analyses.*
- HQ recognized that as a general matter, the currently available commercial finite element codes are not prepared to adequately address some of the complex problems involving ASR-related swelling. In particular, most of these codes lack material models with constitutive relations that are suitable for the description and the evolution of complex material properties related to ASR. Thus, HQ modified a commercial code to address those particular conditions. *NextEra, in contrast, used a commercial code for finite element analysis that was not sufficiently sophisticated nor appropriately modified.* *Id.* at 14-15, 43.
- HQ simulated the behavior of hydroelectric and nuclear plant structures affected by ASR swelling and identified essential inputs to the concrete/reinforced concrete constitutive model accounting for the chemo-mechanical interaction that should be incorporated in advanced Finite Element (FE) codes, which include the following:
    - Adequate description of the kinetics of the reaction;
    - General failure criterion, provision for the development of irreversible deformations, general criterion for the onset of macro-cracking in both compression and tension regimes;
    - Degradation law for strength and deformation characteristics;

- Proper description of propagation of damage in both tension and compression regimes (viz. homogenization incorporating a characteristic dimension, XFEM or similar); and
- Constitutive relation for the interface material relating the velocity discontinuity to the traction vector.

In contrast, NextEra and its consultants failed to list the necessary or desirable features of a code, or its limitations prior to analysis.

- HQ implicitly recognized the need to perform nonlinear analysis, and did not even consider performing a linear elastic analysis. *On the other hand, a nonlinear analysis was not even considered by NextEra.*
- HQ's methodology also included a multi-step and detailed calibration of a large range of factors, using data collected over time. *Id.* at 15-16. As stated by HQ, the step of calibration "is of great importance in any nonlinear static or dynamic analysis as it is a basic requirement for obtaining reliable and accurate results." *Id. In contrast, NextEra's methodology is extremely simplistic and did not consider these many factors.*

Not surprisingly, HQ's far more sophisticated methods yielded a more comprehensive understanding of ASR than obtained by NextEra, and that contradicted NextEra's own conclusions. For instance, HQ found that:

- Humidity distribution plays an important role in determining the long-term, expansion rate and the accumulated total expansion of ASR-affected concrete structures.
- The influence of 1D, 2D or 3D confinement combined with the influence of humidity distribution have to be evaluated based on in-situ measurements of the real structure or based on laboratory tests conducted on concrete samples with sufficiently large dimensions.

Id. at page 13.

And HQ reached a conclusion about the effect of chemical prestressing that is completely at odds with NextEra's conclusion that prestressing is beneficial in the presence of ASR. As stated by HQ:

> The results reveal areas (regions) of the containment with relatively high tensile stresses perpendicular to the planes of the post tensioning cables. The continuous loss of tensile strength of the concrete as a result of AAR may provoke concrete splitting parallel to these planes as it was the case at the Montreal Olympic Stadium (constructed using the same concrete aggregate used at G-2).

Id. at page 28.

A.12 Do you have concerns about the expertise of NextEra's and the NRC Staff's witnesses?

As a general matter, NextEra's and the NRC Staff's witnesses are experienced engineers, and they have demonstrated experience in simple code-based engineering. It is evident that they are more versed in designing (or reviewing) using code-based engineering than in assessing the capacity of an existing deteriorated structure and using/adapting modern techniques. However, as I explained above, code-based engineering expertise is only one of the skills necessary to address a problem as complex as ASR in the aging safety structures at Seabrook. When a structure as critical as a containment enclosure building is affected by ASR, the need to use tools adequate to ensure the safety of the public is compounded.

As best as I can determine none of NextEra's or the NRC Staff's witnesses has demonstrated previous involvement in the specific study of ASR, including recent nonlinear analyses of concrete. Nor have I found any evidence that scientists with ASR expertise were involved in any of the investigations of ASR at Seabrook starting in 2009. The absence of such scientific expertise throughout the investigation and LAR has severely handicapped the LAR process.

Dr. Bayrak's qualifications regarding ASR are also limited. He is a Civil/Structural engineering faculty member with no prior record of accomplishment of research directly related to ASR. His only known previous activity related to ASR was testing large concrete girders suffering from ASR (for the Texas Department of Transportation) with the participation of Prof. Folliard (a leading authority on ASR). However, while Dr. Bayrak may be qualified to conduct large-scale ASR-related experiments, or address code procedures, his testimony does not reflect familiarity with some sophisticated aspects pertaining to ASR (such as impact of relative humidity gradients, impact of reactive sands versus aggregates, inhomogeneity of the ASR reaction within a massive concrete pour and others).

I note that on one occasion, NextEra has indeed solicited the help of an external expert to perform an independent peer review of the evaluation of load amplification factors (for ASR). Prof. Bruce Ellingwood is indeed very well-respected expert in load resistance factor design (LRFD) who has endorsed the proposed methodology. (For the record, I do not disagree with the methodology he endorsed, but instead I disagree with the manner in which it was applied.) It is very regretful that the same attention was not extended to ASR experts to advise NextEra on the multiple ASR-related aspects of this project.

A.13 Do NextEra's and the NRC Staff's experts show sufficient familiarity with the scientific literature that is relevant to ASR at Seabrook?

No. An example of the limited expertise of NextEra's witnesses is provided by a list of "key sources used by NextEra and its consultants" that are "representative of current industry guidance for addressing ASR" (MPR Testimony, A33):

- The Institution of Structural Engineers, "Structural Effects of Alkali-Silica Reaction" (July 1992) ("ISE Guideline") (Exhibit NER012);
- U.S. Department of Transportation, Federal Highway Administration, "Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures" (FHWA-HIF-09-004) (Jan. 2010) ("FHWA Guideline") (Exhibit NER013); and
- Canadian Standard Association International, "Guide to the Evaluation and Management of Concrete Structures Affected by Aggregate Reaction," General Instruction No. 1, A864-00, (Feb. 2000), Reaffirmed 2005 ("CSA Guideline") (Exhibit NRC076).

These documents may be adequate general references for addressing ASR in ordinary structures like bridge piers, but they fall far short of supporting the analysis of a critical safety structure such as the Seabrook containment enclosure building. NextEra completely fails to mention a much larger set of more recent documents that provide more comprehensive and appropriate guidance. To draw such a list would be meaningless in this context (a mere Google scholar search for "Alkali+reaction+concrete+nuclear+dam" yields about 11,000 entries. Of course, only about a hundred would be relevant in this context.) Furthermore, NextEra should have known that there is a wide body of knowledge on ASR coming from the "dam community" that could have inspired it. To name one, the US Bureau of Reclamation's 2005 report, provides very relevant data on the degradation of mechanical properties for structures as old as 30 years.

Similarly, NextEra (and the NRC) should have been very much inspired/guided by the G-2 analysis by H-Q. *See* A.11 above.

A.14 Can you think of current or former DOE/NRC employees who would have had the skills needed to evaluate the condition of ASR at Seabrook?

Abdul Sheikh and Herman Graves (formerly with the NRC) had the skill sets to review such complex structures. Within DOE, Dan Naus (formerly with ORNL), and Yann LePape (with ORNL), or Ben Spencer (INL) have the in-depth understanding of ASR and computational techniques necessary to evaluate ASR at Seabrook.

B Legal and Industry Standards

B.1 In its Statement of Position, NextEra asserts that you would impose requirements on NextEra far beyond what is necessary to satisfy the NRC's reasonable assurance standard. Please comment.

I strongly dispute that assertion. As I have testified in both my Opening Testimony and this Rebuttal Testimony, the work done by NextEra is insufficient to characterize the current condition of ASR at Seabrook or to support a monitoring plan that can identify and assess ASR progression during the 30 years of future operation for which Seabrook has been licensed. As I have repeatedly testified, the simplistic methods used by NextEra to gather data and assess the

condition of ASR at Seabrook are fundamentally inadequate to address such a complex problem with such significant safety implications. NextEra effectively put on blinders to a wide range of available techniques and sources of outside expertise, choosing instead to take a code-based engineering approach that could only scratch at the problem. As a result, the basic safety of the population living within 10 miles of Seabrook cannot be reasonably assured.

I have also suggested a set more modern and effective alternative methods for gathering and analyzing data about ASR (*i.e.*, accelerated expansion tests, periodic damage rating index (DRI) measurements, detailed petrographic studies, and modern computational methods). This approach is not just a different way to do the job, or even just a better way. It is demonstrably effective (for example, Hydro-Quebec), in contrast to the demonstrably ineffective measures used by NextEra.

B.2 Is there agreement among the experts regarding the lack of regulatory or industry standards for ASR in nuclear power plants?

Yes. All of the experts agree that the NRC has no standard that specifically applies to ASR. I also am aware of no regulations or industry standards that have been developed to address specifically the presence of ASR and its implications with respect to serviceability and strength in any field. The often-referenced Federal Highway Administration FHWA document (Exhibit NER013) are guidelines and not standards. A guideline is very different from a standard. A guideline provides recommendations to be accounted for in the specific context of their application. A standard or a code is a must-follow directive. Furthermore, one should always contextualize a document. FHWA document cannot be applied to a CEB with the same assurance as it is applied to a bridge pier. The requirements for a CEB would have of course to be much more stringent. At times, NextEra seems to confuse the two.

B.3 What is the significance to this case of the lack of legal or industry standards for ASR?

It should be made clear that there is no “industry-standard” guidance for ASR (nor for finite element analysis for that matter). As I stated in my Opening Testimony, as a result of the absence of any legal or industry standards for ASR, for all practical purposes it was effectively left to NextEra to write their own guidelines for ASR through their License Amendment Request. This is unusual. As a general matter, guidance documents for evaluating and addressing phenomena like ASR are written by engineering/scientific organizations. For instance, the FHWA report has been written by University Professors/researchers (Prof. Fournier, Berube, Folliard and Thomas). In this case, the NRC’s decision to allow NextEra to write its own standards (without consulting academic professors/researchers in related area to ASR) for testing and analysis of a safety-significant phenomenon that is new to NRC is concerning. The NRC should have done more to

ensure that standards and guidance would be established independently, objectively, and with rigor.

The NRC Staff's testimony is also somewhat misleading by giving the impression that some ASR standards exist. For instance, in NRC-001 the NRC states:

> NextEra has identified reasonable and justifiable structure-specific expansion limits, which account for potential future expansion by setting the maximum level of expansion at which the code acceptance criteria are met, and is actively monitoring all safety-related structures to ensure that they remain within these limits.

This is misleading and of concern as reference is made to "code acceptance criteria" as though there is some industry code that contains acceptance criteria for ASR. It has been recognized by all parties that there is no code for ASR. Indeed, no industry code or regulation contains any maximum level of expansion. Nor does any industry code exist that contains requirements as to how expansion should be measured. Instead, NextEra has proposed to extrapolate the results of the LSTP to Seabrook, despite the dissimilarity between the environmental conditions, concrete mix, in the LSTP specimen and the Seabrook structures. I continue to hold the opinion that what is missing are: accelerated expansion tests on cores recovered from Seabrook (as covered by EPRI Report 3002013192, Exhibit NER018), periodic damage rating index measurements, and more appropriate computational methods to assess safety.

B.4 NextEra has testified that it evaluated parts of its LSTP against the 1971 ACI-318 code. Does that concern you?

ACI-318-71 was written in the "pre-computer" age (1971). It stipulates a linear elastic analysis, and of course it does not mention ASR or alternative analyses methods, and it is a contortionist exercise to use it in the 21st century for such a complex problem as ASR in a CEB. Furthermore, we should bear in mind that the design process is composed of two parts: first, one analyzes the structure to determine the load demand, and then one must examine the capacity of the structure to resist the demand. In the ACI code, there is a dichotomy because one uses linear elastic analysis to determine demand, but one uses a nonlinear (plasticity) approach to determine capacity. In the vast majority of structures, this is acceptable, but for the assessment of the CEB (having a nuclear reactor inside), this approach is highly questionable.

On the other hand, a more recent versions of the ACI code -- ACI 318-14 – which existed at the time NextEra submitted its LAR -- stipulates that inelastic (i.e. nonlinear) Finite Element analysis are permitted for purposes of determining load demand. And in every 21st Century research paper that I have seen addressing ASR and/or cracking, nonlinear finite element analysis is performed. Such an approach should have been followed for Seabrook, just as HQ pursued it for Gentilly-2.

Another major concern, is that the margin of errors in the investigative procedure has not been quantified and is likely to be unacceptable. This makes it too risky to be adopted.

B.5 According to NextEra, "NIST is currently performing a test program for the NRC on ASR that is intended to provide technical data supporting regulatory guidance to evaluate ASR-affected concrete." Of what relevance is this to the Seabrook LAR?

The NIST research, as described in the Regulatory Information Conference (RIC) (https://www.nrc.gov/public-involve/conference-symposia/ric/past/2018/docs/abstracts/sessionabstract-31.html) has had no bearing on this process. It is not referenced in any of the reports prepared by MPR or SGH. Nor is my own research referenced in any of those documents. Furthermore, the NRC Staff does not mention the NIST research or my research in the Safety Evaluation for the LAR.

Of course, this is a matter of concern to me, because it shows that neither NextEra nor the NRC Staff attempted to apply current (NRC-funded) research to the Seabrook LAR or consult outside ASR experts.

B.6 NextEra and the NRC Staff have testified that independent peer reviews were provided by various individuals and entities, including Profs. Ellingwood and Folliard, the Advisory Committee on Reactor Safeguards, and EPRI. Please comment.

With all due respect to the ACRS, there is no indication that they had an appropriate scientific background in Concrete/ASR or that they sought ASR-related expertise outside the NRC. For instance, when the FHWA needed to develop new guidelines, they asked four well-known university researchers (Profs. Berube, Folliard, Fournier, Thomas) to write them. Those guidelines, written for bridges have been extensively referenced by NextEra though a far more important structure is being investigated.

The briefing by EPRI (an industry organization), DOE and the NRC office of research on concrete degradation were very general presentation attended by EPRI, NRC, and DOE employees only. No invited academic speaker attended, including myself (although I was under contract to the NRC at the time.)

As I testified above, Prof. Ellingwood is a very well-respected expert in his field, one of the "fathers" of the load resistance factor design (LRFD) used by the ACI code. But he is not an ASR expert *per se,* although he developed a load amplification factor to the ASR demand. I would also note that his participation in the project was limited to a review of the load amplification factors.

In addition, third-hand reports by NextEra and MPR witnesses regarding the content of telephone conversations with Dr. Folliard simply do not rise to the level of an independent peer review. It is surprising that Dr. Folliard was an active participant in the previous Texas-DOT project, and not

on this one, which could have such extremely serious repercussions. *In-fine,* there is no evidence that he had an input on the project.

In A86, NextEra testifies that it submitted MPR's analysis and recommendation for a large-scale testing program (MPR-3727) to EPRI as an "independent third party reviewer." EPRI is an industry-funded research institute. As such, it is not truly independent of the nuclear industry, including NextEra. Also, unless there have been some recent significant changes in the EPRI staff, it does not have the in-house expertise to fully assess ASR-affected structures. For instance, I recently was contracted to write part of EPRI's report "Structural Modeling of Nuclear Containment Structures" 00-10006428). It is reasonable to assume that EPRI sought my assistance because it did not have sufficient in-house expertise for the task.

With respect to other reviews were performed by NRC staff and employees of the national laboratories, there is no indication that anyone of them had the technical background to fully capture the ASR problem. I note that Dr LePape from the ORNL was not one of the reviewers; nor was Herman Graves, formerly at NRC; or Dan Nauss, formerly at ORNL. All of these individuals would have had the proper technical background for such a task.

C Background: ASR

C.1 In your Opening Testimony, you provided a technical description of ASR. Does it differ significantly from the description given NextEra's and the Staff's testimony, and if so, what is the significance of the difference?

With regard to ASR itself, generally, we all agree that:

- ASR is an irreversible reaction.
- Expansion is limited if constrained (such as by reinforcement).
- ASR is temperature dependent (e.g., the LSTP accelerated the reaction through an increase in temperature).

However, there is a lack of explicit recognition by NextEra regarding the following:

- Relative humidity/temperature is a driver of the ASR reaction (if over 80%) or an impediment (if below 80%). This has an influence on CI readings; see below. NextEra does not account for it in the field measurement of the CI or the subsequent finite element analysis.
- Ignored are the characteristics of aggregates (early or late expansion), ensuing type of gel (in terms of viscosity), or whether the sand or the aggregates are the reactive element. All of the above will influence both the types of cracks (small/larger), the age at which they will develop (early or late). NextEra does not account for this phenomenon.
- The kinetics of the reaction are not accounted for. NextEra makes multiple references to a "slow reaction" and the assumption that the expansion is linear. This is wrong. NextEra

is ignoring the well-established sigmoidal shape of the expansion. Seabrook is most likely in the very early slower phase, but the rate of expansion will accelerate at some point. Hence, the kinetics should play an important role in the LAR (although NextEra repeatedly states that it is irrelevant). Kinetics can be assessed through accelerated expansion tests as described in EPRI Report 3002013192, Exhibit NER018).

- NextEra does not account for degradation of concrete mechanical properties in the Structural Evaluation Methodology (SEM), such as the elastic modulus, or tensile/compressive/shear strengths.
- NextEra does not acknowledge the fact that ASR is not uniform or homogeneous within the CEB walls. Some locations will have more ASR than others. Similarly, "pockets" or "hot spots" are not distributed uniformly. This will result in localized weakness that is difficult to pinpoint and are likely to cause stress concentrations missed by a uniform/smooth distribution of ASR.

D Discussion of Expert Testimony by NextEra and NRC Staff Witnesses

In this section, I will address the most significant points in NextEra's and the NRC Staff's testimony with respect to the following issues: Interim Assessment of ASR at Seabrook (D.1); Relationship Between LSTP, SRP and SEM (D.2); Representativeness of LSTP (D.3); In-Plane Shear (D.4); Chemical Pre-Stressing (D.5); Relative Humidity Implications (D.6); Monitoring (D.7); and Structural Evaluation Methodology (D.8).

D.1 Interim Assessment of ASR at Seabrook

D.1.1 In A81-86, MPR witnesses describe NextEra's program for the interim assessment of ASR at Seabrook. Please comment on their testimony.

MPR consistently refers to the "slow rate" of ASR progression at Seabrook. *See* A80, A81, A83. However, "slow" is not a technical term that applies to ASR. It is well known that ASR follows a sigmoid curve (Figure 17), (Ulm-Larive 2000) and therefore its progress is not linear. Whereas there may be a long latency time, at some point the reaction "takes off" and progresses much faster. This is a very important characteristic of ASR that does not appear to have been considered by MPR or NextEra.

I am also concerned that MPR and SGH In A82 do not account adequately for load distributions. MPR describes the interim structural assessment for Seabrook, stating that net "capacity reduction factors" were determined on a general basis; and that for locations deemed unacceptable, "conservatisms in the demand (i.e., loads and load factors" were applied. SGH also discusses ASR-related load in A53-A56.

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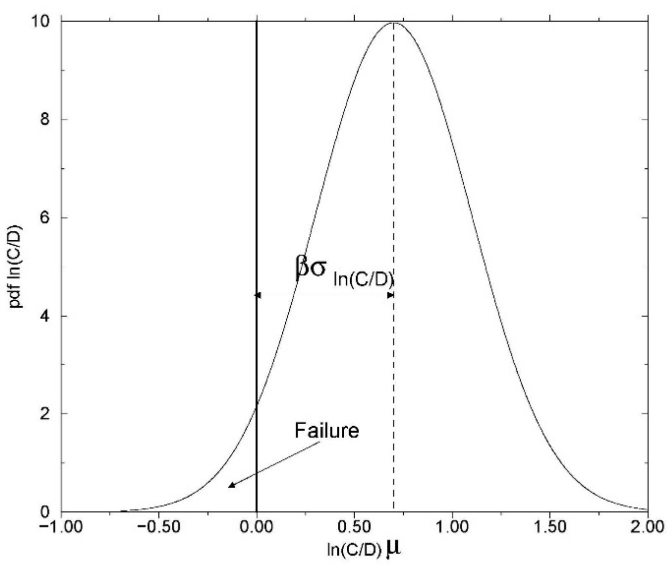
Whereas the Load Resistance and Factor Design (LRFD) method governs the design of new structure, when applied to check the adequacy of a structure, it will result in a binary outcome (pass or fail). Furthermore, by virtue of the linear elastic analysis, the displacements are grossly underestimated (and thus cracking cannot be predicted). As such, the Federal Emergency Management Agency (FEMA) has encouraged the use of the so-called Performance Based Earthquake Engineering to address partial failures, and load redistributions. However, this would have required a nonlinear analysis. (FEMA P58, Seismic Performance Assessment of Buildings. Though this analysis paradigm was not available when Seabrook was built, it constitutes the most rational engineering practice method to assess the safety of Seabrook (ironically, the method has its root in NRC studies).

D.1.2 <u>In A59, SGH's witnesses claim that in the course of applying ACI 318-71, they "did account probabilistically for the variation of ASR across the plant." Do you agree?</u>

I agree only to a limited extent. SGH's method does recognize the probabilistic nature of both capacity and demand, and set the factors such that the reliability index (defined below) β is around 3.5. Yet, the investigation of ASR at Seabrook was, fundamentally, deterministic. I have previously described the deterministic character of NextEra's overall approach and why it is insufficiently sophisticated to address the complexities of ASR in the CEB.

To a very limited extent, and in an implicit way, SGH did use probabilistic analysis (as embedded in the LRFD method is the recognition that both demand and capacity have a probability distribution function, Figure 1. I nevertheless have overall concerns. First, the load factors for ASR, while they were determined by an appropriate method, rely heavily on field measurements of the CI, which – as I have previously testified -- are grossly inadequate indicators of ASR.



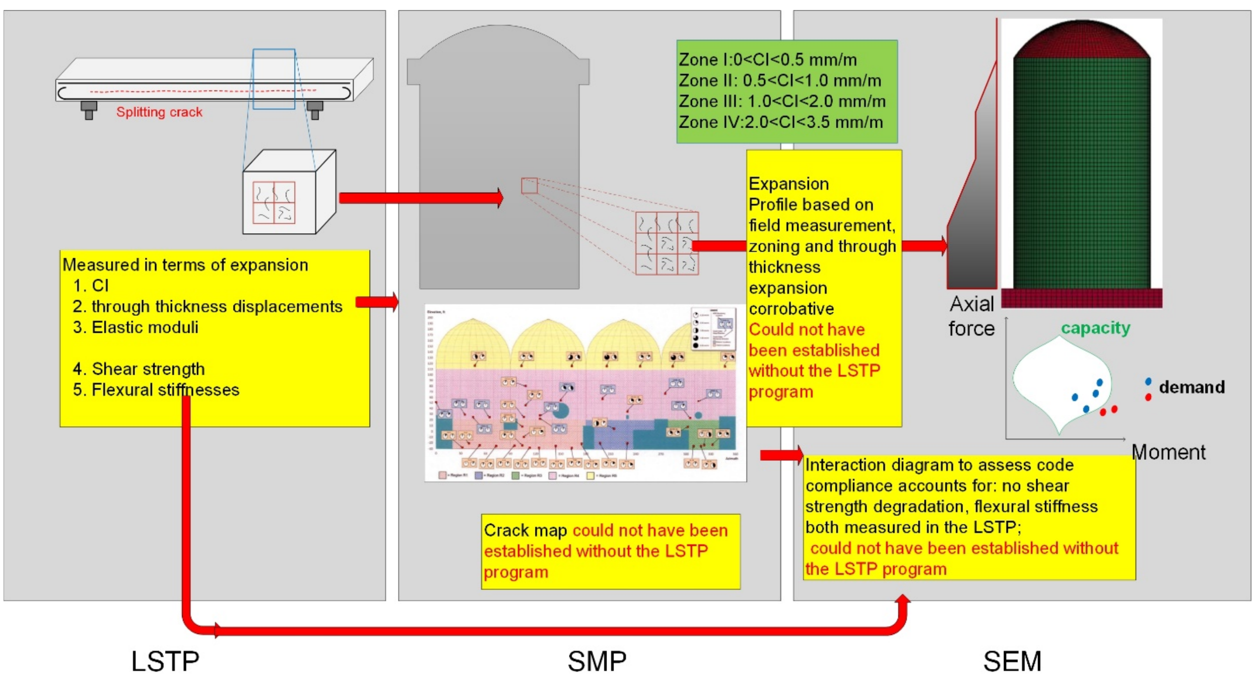
<u>Figure 1 Normal distribution of capacity over demand in the definition of the reliability index</u>

Second, the ASR load factors are amplified by a threshold of 1.2 to account for additional ASR loads that may occur in the future. I have found no explanation of how the 20% increase was determined. Adding a conservatism is appropriate, but not where the purpose is to correct for the essential unreliability of CI to detect ASR. In addition, if the 20% margin was added to account for unknown changes in the ASR growth rate, that is also a factor that could have been assessed with much greater precision. In sum, conservatisms can never overcome fundamental inadequacies in data or analyses.

D.2 Relationship between LSTP, SRP and SEM

D.2.1 <u>In A90, MPR's witnesses provide a flow chart that shows the relationship of the LSTP to the Structures Evaluation Methodology and the Structures Monitoring Program. You have also testified regarding the dependence of the SEM and the SMP on the LSTP results. Do you have any further comment?</u>

Yes. Figure 1 below illustrates the tight integration of the LSTP, SEM, and SMP.



<u>Figure 2 Tight integration of LSTP, SM and SEM</u>

The LSTP performed numerous measurements, all in terms of expansion, that fall in two different categories:

1. Material characteristics that facilitated the subsequent SMP and SE:
   a. Crack index

b. Through thickness displacements
c. Elastic moduli

NextEra developed, deployed, and validated an approach that allows the determination *in-situ* (at Seabrook) of in-plane, and out of plane expansions. Those measurements will be taken *in-situ* from the SMP (field measurements) to the SEM (for the finite element analysis safety assessment). Such an innovative technique, never tested before, had to be first validated in the laboratory. This was done through the LSTP. <u>Hence, the LSTP had a direct impact on the SMP and SEM.</u>

2. Structural component characteristics that affected the subsequent SE:
   a. Shear capacity
   b. Flexural stiffnesses.

For the final safety assessment, the finite element analysis first determines the demand (based on the various loads), and then the capacity (ability of the structure to resist a particular load under certain conditions, ASR in particular). Capacity under those conditions not being addressed by codes, NextEra has performed large-scale tests (LSTP) to determine how ASR would affect the shear resistance (it was found to increase with ASR). As a result, the finite element models were accordingly adjusted. <u>Without this observation, the SEM would not have been possible.</u>

Hence, without the LSTP, the SMP and SE would not have been achievable.

D.3 <u>Representativeness of LSTP</u>

D.3.1 <u>In A109 - A111, MPR's witnesses agree with you that reinforcement dimensions and configurations are important in evaluating the ability of ASR-affected concrete to resist expansion. They say the LSTP considered those factors. Please comment.</u>

NextEra alleges here, and in previous statements, that the tested beam is representative. That is simply incorrect. The reinforcement ratios along the longitudinal axis x and the transverse on y are [redacted] (using the [redacted] and not [redacted] bars [redacted]) and [redacted] respectively. The reinforcement ratio in the y direction is not consistent with the one in the x direction. I do not know what is the exact ratio at Seabrook, but usually it is around 0.5%.

In addition, and most importantly, MPR-3848 (Exhibit NER015) states that the test specimens were intentionally fabricated to <u>different</u> specifications in order to induce supposed conservatisms:

> The specimens for the Shear Test Program will have a higher reinforcement ratio than the reference location to ensure that the failure mode is shear. The higher reinforcement ratio will provide higher in-plane restraint, which will promote greater expansion in the out of plane direction-the direction that will have the greatest impact on shear

performance. Hence, using the in plane CCI to translate between test results and structures at Seabrook Station will be conservative.

D.3.2 <u>In A40, the NRC Staff asserts "the LSTP was not a model test; rather, . . . it was a full-scale load test. It remains a scaled test." And in A29, the NRC Staff states that "the LSTP showed that, for heavily loaded members (i.e., members with flexural cracking), the flexural stiffness increased as ASR expansion increased within the expansion levels tested." Please comment.</u>

To begin with, if the beam is flexurally over-reinforced, one cannot assume that for Seabrook Station the same effect will be observed. It may, or it may not. One has to make a more convincing argument to support such a comparison.

Another important difference between the test specimen and the CEB is that the test specimen was about [redacted] scale ([redacted] depth whereas the wall of a CEB is about 36 inches). This is not unusual in component testing. However, given the brittle nature of shear failure and associated size effect, the shear strength in the CEB will be lower than the one from the LSTP. This phenomenon is described by Figure 3, a depiction of the LSTP derived from Bentz, E.C. (2005).

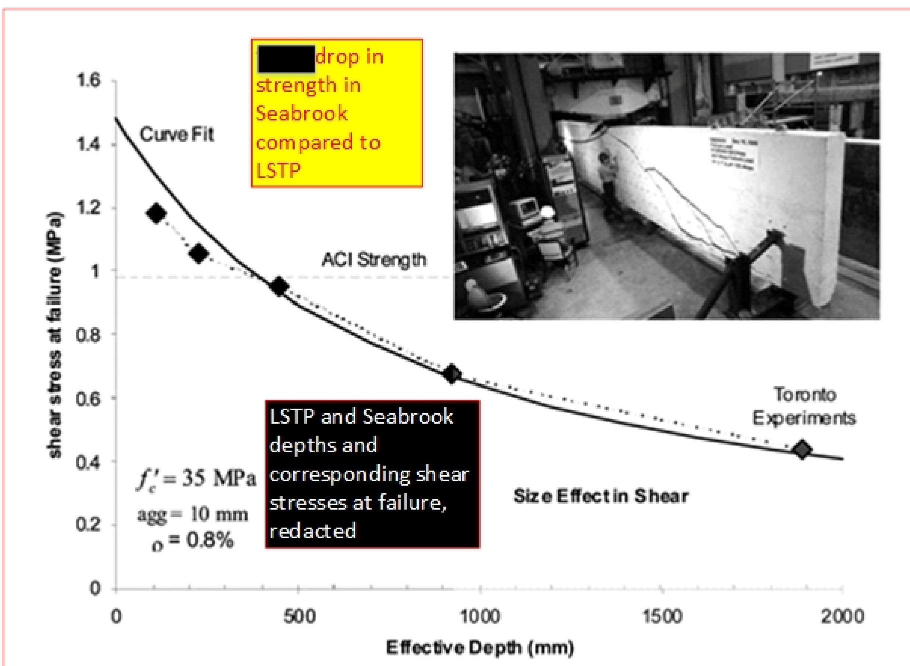


Fig. 1—Example of size effect in shear for reinforced concrete members without stirrups.[1]

<u>Figure 3 LSTP is a model test and not a prototype test, there will be a [redacted] reduction in shear strength in Seabrook. Adapted from Bentz (2005).</u>

The LSTP is [redacted] deep, whereas the CEB at Seabrook (in its most critical location: base) is 36”. Hence, the LSTP is indeed a model test. Furthermore, due to size effect, the strength of a 36” deep beam (modelling the CEB wall) is about [redacted] lower than the one of a [redacted] one.

Having said that, ultimately what is the actual strength of the beam is irrelevant as this value is not used in the subsequent analyses. Ultimately, I consider that all those shear tests were useless as results were qualitatively known and could have been quantified (at a much lower cost) through proper finite element studies. Furthermore, the only impact they had on the two subsequent tasks was to experimentally confirm the lack of shear strength degradation of a reinforced concrete beam.

Whether or not the shear tests were important, I remain deeply concerned about the lack of rigor of the investigation, in which such a major structural crack was disregarded.

D.4 In-Plane Shear

D.4.1 In A40, the NRC states that “the LSTP did not test for the in-plane shear mode. This was because the out-of-plane shear failure mode was judged to be more critical than in-plane shear mode (note: nominal permissible out-of-plane shear stress in concrete per the ACI 318-71 code is 2*√f’c versus allowable total shear stress of 10*√f’c for in-plane shear (NRC049 § 11.4.1 at 37, § 11.16.5 at 42); here f’c is the specified minimum concrete compressive strength).” Please comment.

The fact that the ACI 318-71 code allows 10 times the square root of the compressive strength for in plane shear, as opposed to only two times for out of plane, is irrelevant. In both cases, the relative loss in strength will be equal to the square root of the fraction of the loss because the 2 and the 10 cancel out). For instance, if the original compressive strength is 100 (never mind the units), and due to ASR the compressive drops to 70, the loss in shear strength for both in-plane and out of plane will be equal to the square root of 70 divided by 100 (0.83).

Hence, the concrete deterioration of the in-plane shear should be accounted for. This is important, as the analysis of the container is not accounting for this loss.

D.4.2 In A29, the NRC Staff asserts that the LSTP addressed seismic response and flexural stiffness in a manner that was representative and/or bounding of ASR at Seabrook. In making this assertion, the Staff also states that:

> *The natural frequency of a structure is proportional to the square root of the stiffness to mass ratio. Based on a view of the LSTP data, the Staff noted that for heavily loaded structures (i.e., members with flexural cracking), the flexural stiffness increased as ASR expansion increased within the expansion levels tested and … since an increase in stiffness will increase a structure's frequency (considering no change in mass), it is reasonable to conclude that ASR will not have an adverse impact on seismic response for heavily loaded structures; therefore, the Staff found it acceptable for NextEra to conclude that the seismic analysis in the current licensing basis described in UFSAR Section 3.7 remains valid (NRC007).*

Do you agree?

No. Under seismic excitation, only about half of the CEB is resisting the lateral load through flexure. The rest is in near pure in-plane shear. In that zone, one cannot rely on the flexural stiffness, but instead one must rely on the shear modulus, which – as discussed above -- decreases due to degradation.

D.5 Chemical Prestressing

D.5.1 In A68 – A70, MPR's witnesses describes the "chemical prestressing effect" in reinforced concrete. They assert that chemical prestressing can "offset a certain level of ASR-induced degradation" and cause "an increase in shear strength." Do you agree that the chemical prestressing effect is fundamentally beneficial?

No. While MPR states that "the beneficial effects of confinement are recognized in the structural engineering community," its potentially adverse effects are also recognized in Exhibit NRC075, Dean J., Deschenes, et. al.,2009) summarizing work conducted for the Texas Department of Transportation at Ferguson Structural Engineering Laboratory, The University of Texas at Austin (August 2009). As discussed in Exhibit NRC075 at pages 25-26, more than thirty cases of "fractured reinforcements" have been found in bridges and other structures.

The phenomenon is simply illustrated below.

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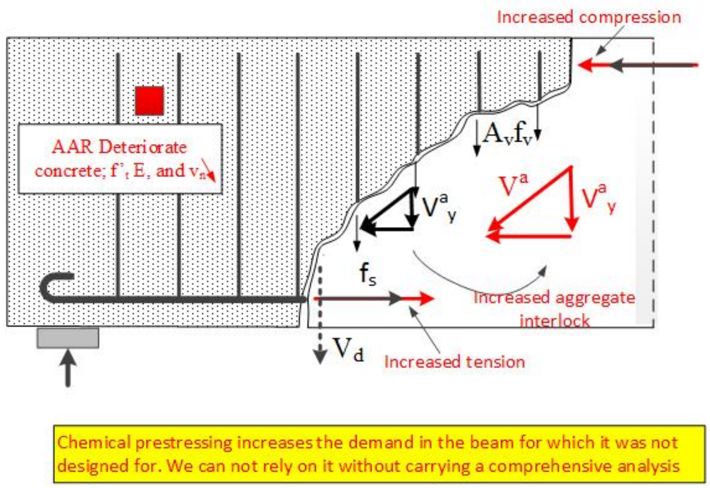

Figure 4 Consequences of chemical prestressing

As demonstrated in Figure 4, chemical prestressing increases the demand in the beam for which the beam was not designed. Thus, the beam cannot be relied on without undertaking a comprehensive analysis.

The real possibility of excessive steel stresses resulting in premature fracture or yielding was also reported (Miyagawa et al. 2006). Fracture of Reinforcing Steels in Concrete Structures Damaged by Alkali-Silica Reaction. Journal of Advanced Concrete Technology, 4(3), 339-355. Indeed, in this paper, it is shown that “chemical prestressing” has caused the rupture of steel and thus partial collapse of a bridge. *See* Figure 5 below.

Figure 5 Picture of fractured reinforcement as a result of ASR expansion (Miyagawa, et al. 2006).

The reported rupture has triggered much research in Japan such as "Effect of Rupture of Shear Reinforcement on Shear Capacity of ASR Damaged Reinforced Concrete Beams by Inoue et al. (2008) which states: "In recent years, it is reported in Japan that stirrups, as well as longitudinal steels, in T-shaped beams of bridge piers were ruptured at the bent corner or butt joints. In order to make clear the causes of this rupture, vigorous research works has been done after the finding of the rupture in existing reinforced concrete structures. Up to the present, it is recognized that this phenomenon occurred not only due to excessive ASR expansion but also under complex combinations of several factors, such as mechanical properties and surface shape of reinforcing bars, bending or welding methods of reinforcing bars, corrosive atmospheres and so on."

Given that NextEra's experts have not recognized this potential rupture mode (although it is cited in Exhibit NRC075 page 25-26 that they rely on) I am concerned that it may not be captured in their analysis. My concerned is heightened by the fact that the effects of chemical prestressing are analyzed in the relatively simplistic context of linear elastic analysis.

The shear beam should have had strain gags placed on the reinforcements to asses and quantify the adverse effects of the chemical prestressing (undesirable effect of additional stresses that could have prematurely yielded or fractured the rebars in Seabrook). This was not done even though it appears from Prof. Bayrak testimony (Exhibit NER019, page 16) that he was aware of this potential problem.

D.6 Relative Humidity: Implications

D.6.1 What is the role of relative humidity in ASR?

Relative humidity (RH) plays a critical role in the expansion of concrete. As mentioned in our previous reports, a RH less than 80% will not result in any expansion. This has very strong implication on the surface measurements of cracks to ascertain the presence/extension of ASR. In other words, due to shrinkage and external exposure, the surface of the wall most certainly has a relative humidity less than 80%. As such, one would only capture severe cracks emanating from the center and that made it to the surface. Those are what I will refer to as ASR cracks, not to be confused with internal structural cracks that I will address separately. The implication being (and this will be again discussed in the next question) that the CI will not be reliable.

For the ASR to occur the relative humidity has to be at least 80% (Stark, et al., 1987). This is implicitly recognized in the report (Sect. 4.2.6 of the LAR (Exhibit INT010)) as the test specimens were stored in an Environmental Conditioning Facility (ECF) because "*ASR proceeds more rapidly in hot and moist conditions.*"

Furthermore, the higher the temperature, the faster the expansion will be.

Finally, it should be emphasized that, unlike the consistent temperature and humidity conditions created for the beams in the LSTP, neither the relative humidity nor the temperature is constant across the 36" CEB wall. This is schematically illustrated by Figure 6.

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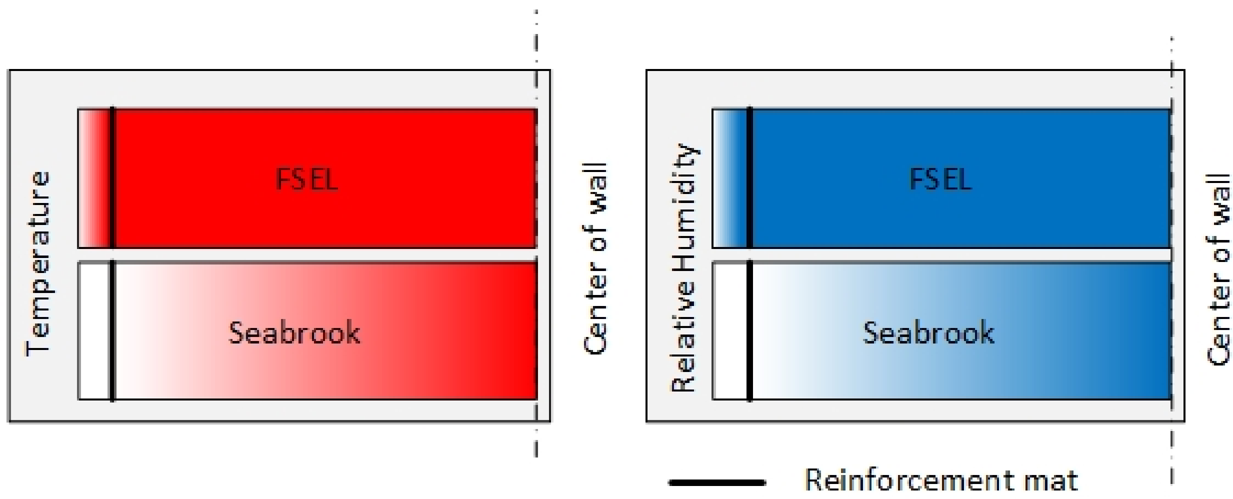

Figure 6 Factors affecting progression of ASR

The following table highlights the different environmental conditions:

LSTP	Seabrook
Temperature kept high to simulate ASR expansion. Given the dimensions of the beam, and the internal heat of hydration, one can reasonably state that the temperature was uniform (no gradient) across the beam.	There is always a temperature gradient across the 36" wall.
The relative humidity was kept high in the FSEL (by covering the specimen with burlap. Because of the continuously wetted burlap and the high water to cement ratio, it is safe to state that in the FSEL the relative humidity was constant across the specimen. *	The surface of the wall has dried, and is not prone to expansion whereas the expansion will take place inside the wall where the relative humidity is much higher.
The FHWA report stipulates that for CI measurements, the "most severely cracked" components "generally correspond to those exposed to moisture and severe environmental conditions, as well as those where ASR should normally have developed to the largest extent." Exhibit NER013 at page 12.	The surface of the CEB is certainly no longer moist, it has dried.

"The internal humidity of the concrete and the atmospheric conditions in the ECF were sufficient to drive progression of ASR uniformly throughout the test specimens." Exhibit INT019, MPR-4273, Revision 1, "Seabrook Station - Implications of Large-Scale Test Program; Results on Reinforced Concrete Affected by Alkali-Silica Reaction" at page 4-8 (July 2016).

[redacted]

Figure 7 Specimens covered with burlap in the FSEL (Exhibit NER022, MPR-4262 (PROPRIETARY)

Hence, the ideal conditions in the LSTP which were intended to validate the CI measurements' reliability for Seabrook are not representative of in-situ conditions at Seabrook.

This has strong implication on being able to capture internal cracks by surface measurements.

The low RH on the surface of the wall, compounded with the proximity of the large mat reinforcement (number 11 bars) clearly inhibit the formation of surface material ASR cracks (in other words the reinforcement will "pinch" the crack and the opening on the surface will be much smaller than on the inside). This is further by illustrated in Figure 8.(where the crack opening is in red)

In light of the above, I continue to insist that the expansion inside Seabrook CEB is almost certain to be much higher than what can be recorded by CI.

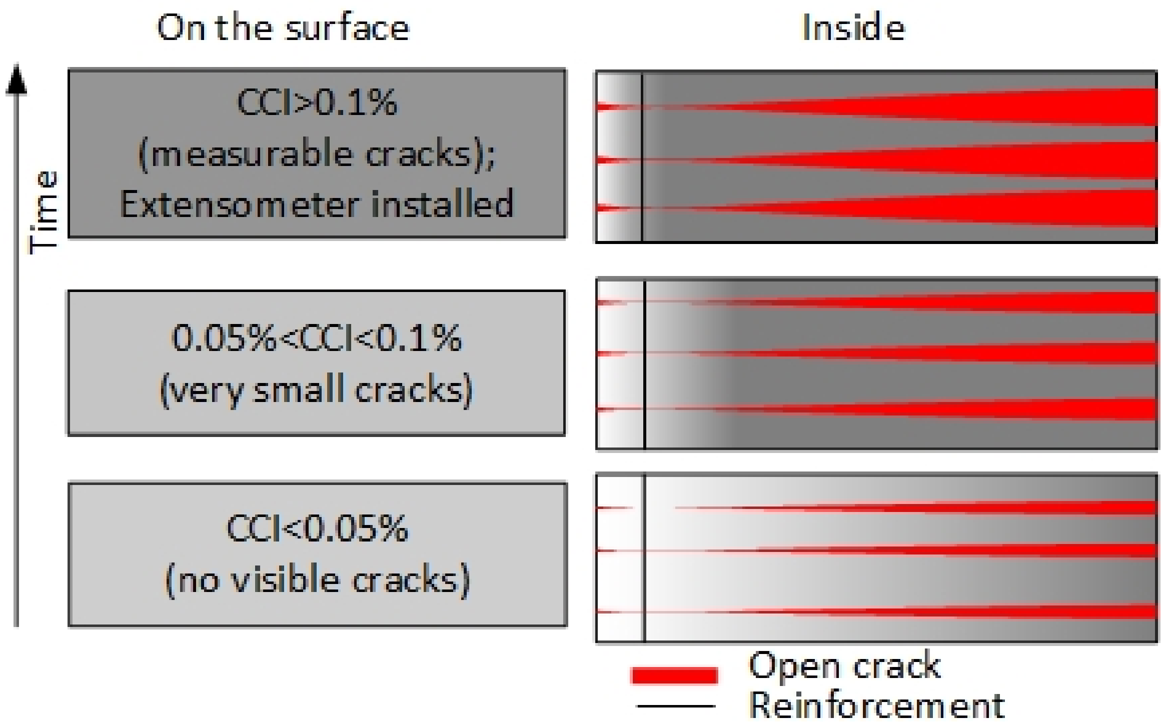

Figure 8 Possible scenario of internal cracks formation and unlikelihood of being captured in time by CI readings

D.6.2 In A73, MPR's witnesses acknowledge that water plays a role in ASR progression. And in A214, they agree that a relative high level of humidity is a significant factor in ASR progression. But in A214, the MPR witnesses disagree with you about the need for internal monitoring of humidity as a means of projecting future ASR expansion. They say it is not necessary, because ASR is *assumed* to exist at Seabrook and "the only need for understanding rate of expansion at Seabrook is validation that the monitoring frequency is sufficient." They say that "NextEra is using in-situ monitoring for this purpose." MPR also asserts that your testimony "focuses on the need for understanding the potential for future expansion," but "the ASR monitoring methodology at Seabrook does not rely on such projections for long-term, or even medium-term expansion." According to MPR, "the only need for understanding rate of expansion at Seabrook is validation that the monitoring frequency is sufficient, and NextEra is using in-situ monitoring for this purpose." Please comment on their testimony.

In A73, the MPR witnesses do not recognize the role of internal relative humidity in the ASR expansion. A73 simply refers to "water." What they may have failed to say is that having a surface

exposed to water will stimulate the ASR reaction because through osmosis the concrete will develop internally a high relative humidity.

In A214, MPR tries to avoid directly addressing the role of relative humidity. The fact that "NextEra's SMP assumes that ASR exists in all plant structures" is irrelevant to the question of whether relative humidity plays a role that should be considered. MPR's suggestion that I have improperly focused on the need to understand the potential for future expansion is also irrelevant. The point is, the MPR witnesses utterly disregard the fact that unless the (measured) internal humidity is above 80%, there simply cannot be any ASR expansion. Which is another reason to seriously question the reliability of the CI method.

D.6.3 In A210, MPR's witnesses say that the FHWA Guidelines (Exhibit NER013) endorse CCI as a tool for measuring ASR, and that it also warns against relying on the Damage Rating Index (DRI) methodology. Please comment.

As I have previously testified, the FHWA report was written for bridges and not a 36" thick dry wall enclosing a nuclear reactor. All codes differentiate the types of structures, and assign different factors of safety accordingly to the occupancy. Though not explicitly addressed in the 1971 code the 2014 one (and the Uniform Building Code) have different criterion for seismic safety based on the structure occupancy. Hence, requirements for a hospital or a school will be much more stringent than the one for an isolated farm.

Likewise, the same principle applies here. The FHWA Guidelines were written for bridges which can be easily visually inspected and whose eventual failure cannot be as catastrophic as a (localized) failure in a CEB. In other words, it would be inappropriate to apply the very same limits set by the FHWA to the containment building of a nuclear reactor. Requirements must be more stringent.

The FHWA Guidelines themselves acknowledge that they may not be adequate for some applications. For instance, in Section 2.2 (at page 3), the FHWA Guidelines state: "The quantitative assessment of the extent of cracking through the Cracking Index, along with the Petrographic Examination of the cores taken from the same affected element, is used as tools for the early detection of ASR in the concrete." This statement implies that CCI would not necessarily be adequate for a safety investigation of the impact of ASR over the long-term, and may necessitate a Level 3 investigation.

As discussed in Section 2.3 (page 4), a Level 3 investigation entails: "An in-situ investigation program which includes monitoring of expansion and deformation generally provides the most reliable "prognostic" for ASR-affected structural members. Considering the seasonal variations in climatic conditions that affect the progress of ASR and the differences in the reactivity levels of aggregates and other mix design considerations (alkali contents, etc.), it is generally considered that a minimum of 2 years and ideally 3 years are required for reliable decisions on the implementation of remedial actions to be drawn from in-situ monitoring programs. A reasonable estimate of the potential for further expansion/deterioration can also be obtained through a

detailed laboratory testing program. Such a program involves a series of tests on cores extracted from the concrete member/structure investigated, as listed in Table 1. In most severe cases of deterioration, an assessment of structural integrity may be required. The above investigations will provide further critical information in the selection of repair and/or mitigation strategies." (Emphasis added)

The FHWA Guidelines also provide that if the extent of cracking is considered "important" (i.e., the CI is greater than selected criteria) and definite petrographic signs of ASR are noticed, "additional work may be required (i.e., ASR Investigation Program Level 3) and/or immediate remedial actions can be applied. Hence, the FHWA places severe limitations on the applicability of the CI method, which are not necessarily respected in this investigation.

To summarize, given the potential gravity of the situation, merely adhering to a code written for bridges without due examination to alternatives is troubling. We maintain that the CCI method is not a reliable measure.

D.6.4 In A31, the NRC Staff states "Cracking is always more pronounced on the free surface of a structure or component because that is where it is most free to develop and grow." Do you agree?

The Staff's testimony ignores the fact that on the surface, we have dry concrete and we are close to the reinforcement that constrains its opening. Most importantly, NextEra assumes that ASR will cause radial cracking, while acknowledging that the out of plane expansion is predominant. This is inconsistent: most of the crack will be internal along the lines of compression and will barely daylight.

D.7 Structural Crack: Implications

D.7.1 You expressed great concern about the structural crack caused by ASR in the shear beam tests. Why?

Indeed, it is intuitive that ASR being a volumetric expansion, and in the presence of X and Y restraint, most of the expansion would be in the vertical direction (Z axis). As the beam is unreinforced in that direction, a crack will naturally form.

Now it is important that we understand what a crack means in cementitious materials (as in concrete). For that, we need to mention the well-established model of Hillerborg 1976), which breaks a crack into two parts: a) fracture process zone (FPZ) and b) true crack. Along the FPZ the tensile strength decreases with the crack opening. At a critical opening (which is a fraction of a mm), we will have a true crack (Figure 9). Here, contrary to metal, it is nearly impossible to observe the tip/front/surface of a FPZ; however, it can be indirectly measured.

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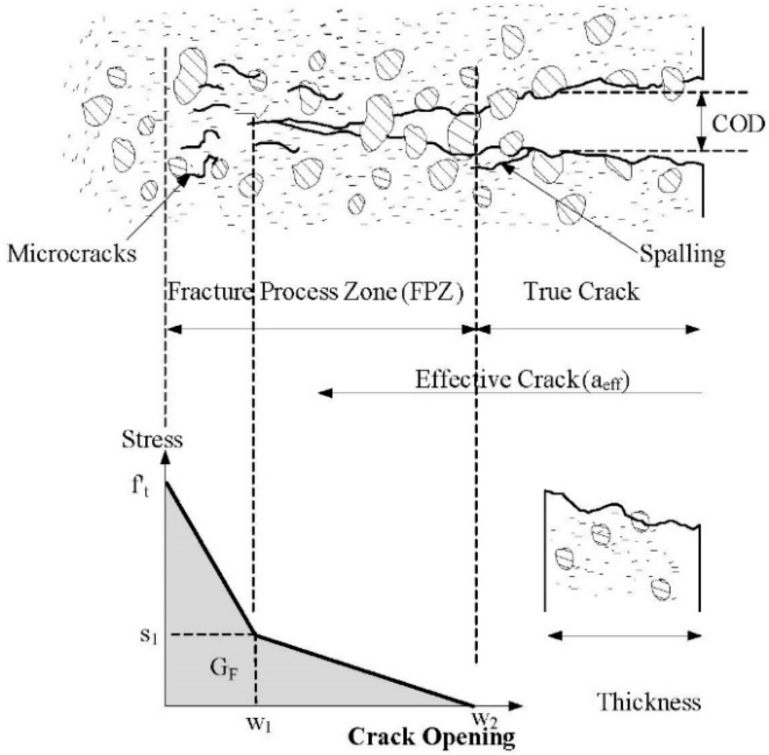
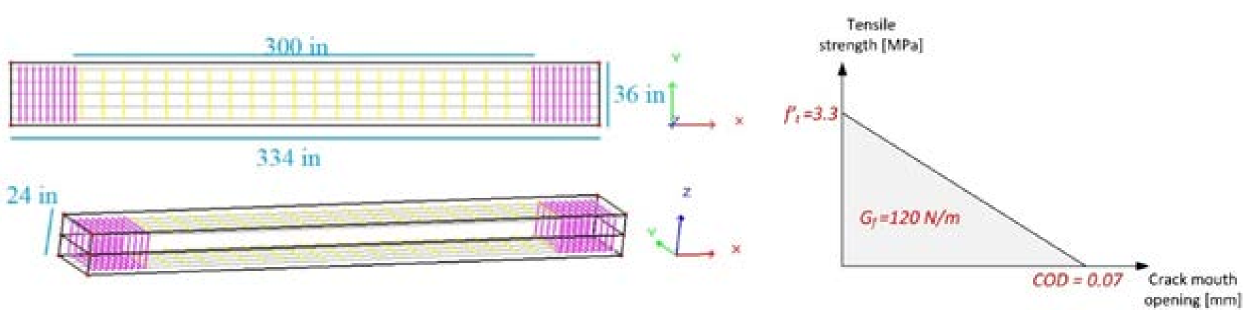


Figure 9 Hillerborg's model for concrete (adapted from Hillerborg 1976)

Hence, to better understand what has happened a nonlinear 3D fracture mechanics based finite element analysis of the tested beam was conducted (Figure 10),



a) b)

Figure 10 a) Finite element model analyzed to duplicate pre-test cracks (From Wald, David, Gloriana Arrieta Martinez, and Oguzhan Bayrak. "Expansion behavior of a biaxially reinforced concrete member affected by alkali-silica reaction." **Structural Concrete** ***18.4 (2017): 550-560.; and b) adopted cohesive crack model***

Results are summarized below

1. Because of unconstrained vertical expansion, indeed a crack formed in the portion of the beam that is without shear reinforcement (stirrups) as shown in Figure 11. This matches the crack observed in the LSTP, Figure 13.

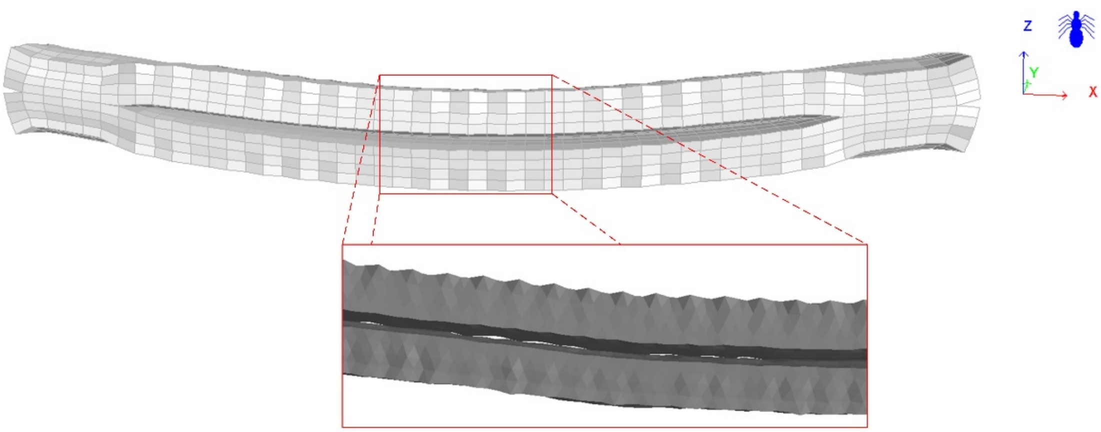


Figure 11 Highly amplified deformation showing crack opening in between the stirrups

2. At midspan, the computed crack profile is shown in Figure 12. It matches what has been reported by NextEra that is the crack opening is largest on the edge. We estimated it to be 1.4 mm (this may be less than what was determined, but again in the absence of tests we had to estimate the fracture energy GF).
3. The close-up showing an "oscillation" in the vertical displacements is due to the confinement provided by the transverse rebars (along the Y-axis). In other words, displacements are constrained by the bars, but in between, we have small spikes.
4. We also have the crack profile showing a minimum in the center (about 0.3 mm). Again, this (to some extent) matches what NextEra has "dubbed" an inconsequential edge crack.
5. Because the minimum crack opening is larger than the maximum crack opening beyond which the concrete loses its ability to carry tensile stresses (in accordance with Hillerborg's model, Figure 9) the normal stress is zero.
6. It is not surprising that NextEra did not "see" or detect a crack when they examined the specimen, it is simply that it was much smaller than what they may have anticipated.

Again, it should be emphasized that this analysis does not pretend to be quantitatively exact, however it is most certainly qualitatively correct and based on sound, established, fracture mechanics models.

NextEra indicates that this will not occur in the CEB. This statement is unfounded. Given the lack of reinforcement across the thickness of the CEB, a delamination crack (similar to Crystal River) is in the realm of possibilities.

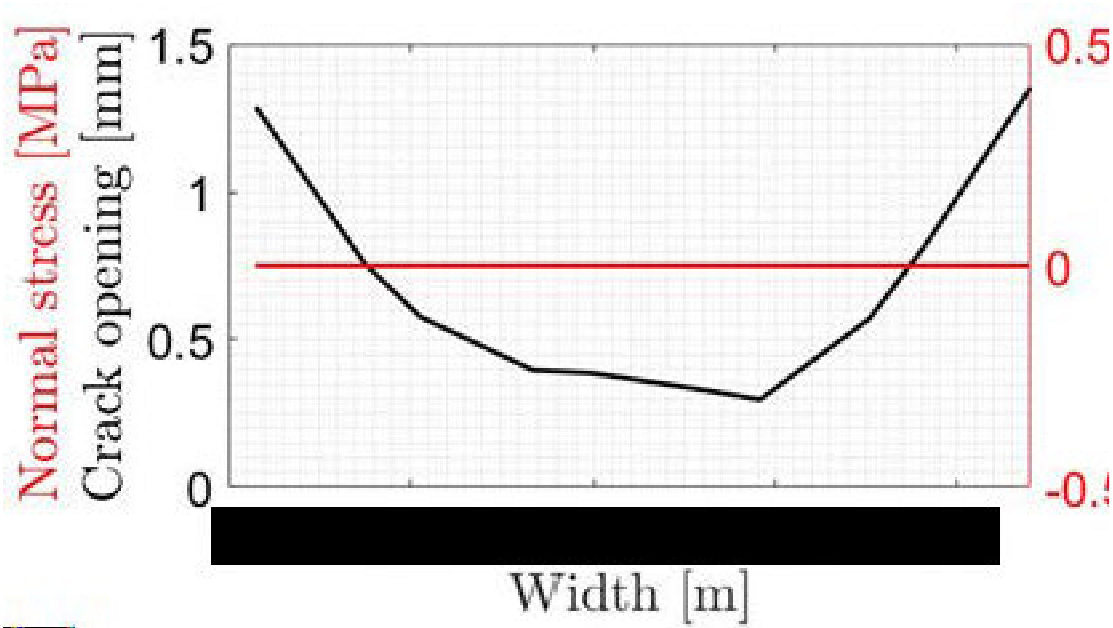


Figure 12 Crack opening profile across the beam (along y axis)

The impact of this crack is two-fold:

1. It may have impacted the validity of the shear tests as in Figure 13 the shear crack trajectory is affected by the presence of this ASR crack.
2. Most importantly, such a crack (delamination), and contrarily to NextEra's assessment may form inside the walls of Seabrook.

[redacted]

Figure 13 Reported Main ASR crack, and shear cracks **[PROPRIETARY]**

D.7.2 What are the limitations of the observed ASR crack in the LSTP on Seabrook?

The previous question highlighted the power of an ASR-induced unconstrained expansion in cracking concrete.

An analogous phenomenon may indeed occur at Seabrook. In addition to the small cracks caused by ASR, we may have a major internal structural crack in-between the two reinforcing mats. Let me explain:

1. ASR is very "opportunistic;" it will expand in the direction of least resistance. In Seabrook, it will mostly (if not entirely for all practical purposes) expand radially, through the unreinforced thickness.
2. Concrete is a very heterogeneous material, in the sense that pours (of concrete) will come from different mix batches, that each one of them may use aggregates that have slightly different reactivity, and that as such there will be "hot-spots pockets" with larger expansions. As a result, ASR expansion will not be uniform inside the walls.
3. Eventually, there will be one (or more) "weak links," i.e. zones whose expansion is such that small structural cracks nucleate. Focusing on one of them, such a crack in turn will propagate with expansion. Indeed, by now, under so-called Mode I (in fracture mechanics), all the elements are present to have this crack propagate along the line of high compression (that is vertically). Once expansion reaches a point where the (degraded) tensile strength of the concrete has been reached, we will have a crack nucleation.
4. The next concern is whether the crack will propagate. It will, because it is driven by the increased swelling of concrete.
5. Once we have a crack, this does not cause the ASR induced expansion to stop. It will continue and will cause the crack to propagate. As shown in Figure 14, there are three factors contributing to the propagation of the internal planar crack:
    a. The expansion is out of plane (radial) because it is constrained in the other two directions.
    b. The wall is under compression, and therefore cracks (no matter how small) will propagate vertically along the line of compression.
    c. Because of the axial longitudinal compression, due to Poisson's effect, there will be a tensile stress in the orthogonal (radial in this case) direction.

   We thus have the "perfect storm" to cause an internal delamination once the expansion has reached a critical stage.

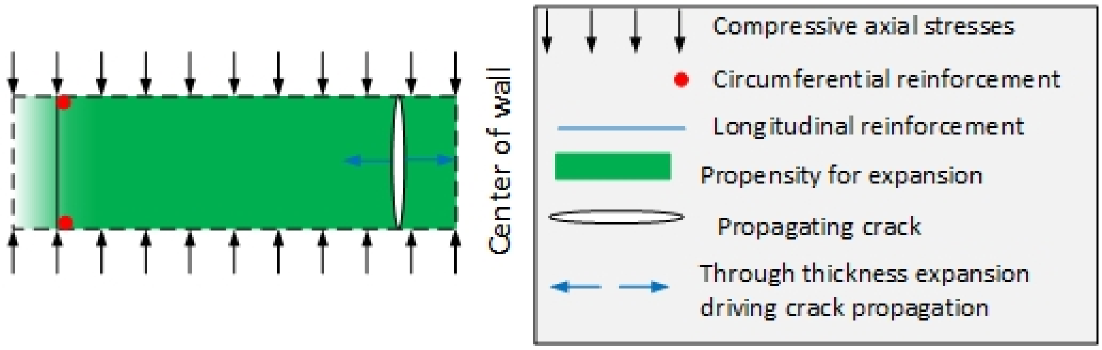


Figure 14 Mechanism of internal delamination

6. At some point, those cracks will coalesce (i.e. inter-connect) causing internal delamination, Figure 15. This will be an internal structural crack, causing delamination (through the very same mechanism that caused the cracking of the shear beam even before being tested, with the additional complication of having a compressive stress that will dope it even more). This delamination will severely weaken the CEB. Most importantly:
   a. The crack will not be capable of detection by surface measurements (CI).
   b. The delamination is unlikely to be captured by an extensometer because of the "patchy" nature of ASR hot-spots or pockets, and because there may not be corresponding surface in-plane cracks detected by the CI method. (Remember that ASR is not homogeneous within the walls. Thus, failure to capture that internal crack with extensometers, does not mean that the crack is not "sleeping" inside the wall.)

EXHIBIT INT028

~~CONTAINS PROPRIETARY INFORMATION – SUBJECT TO PROTECTIVE ORDER~~

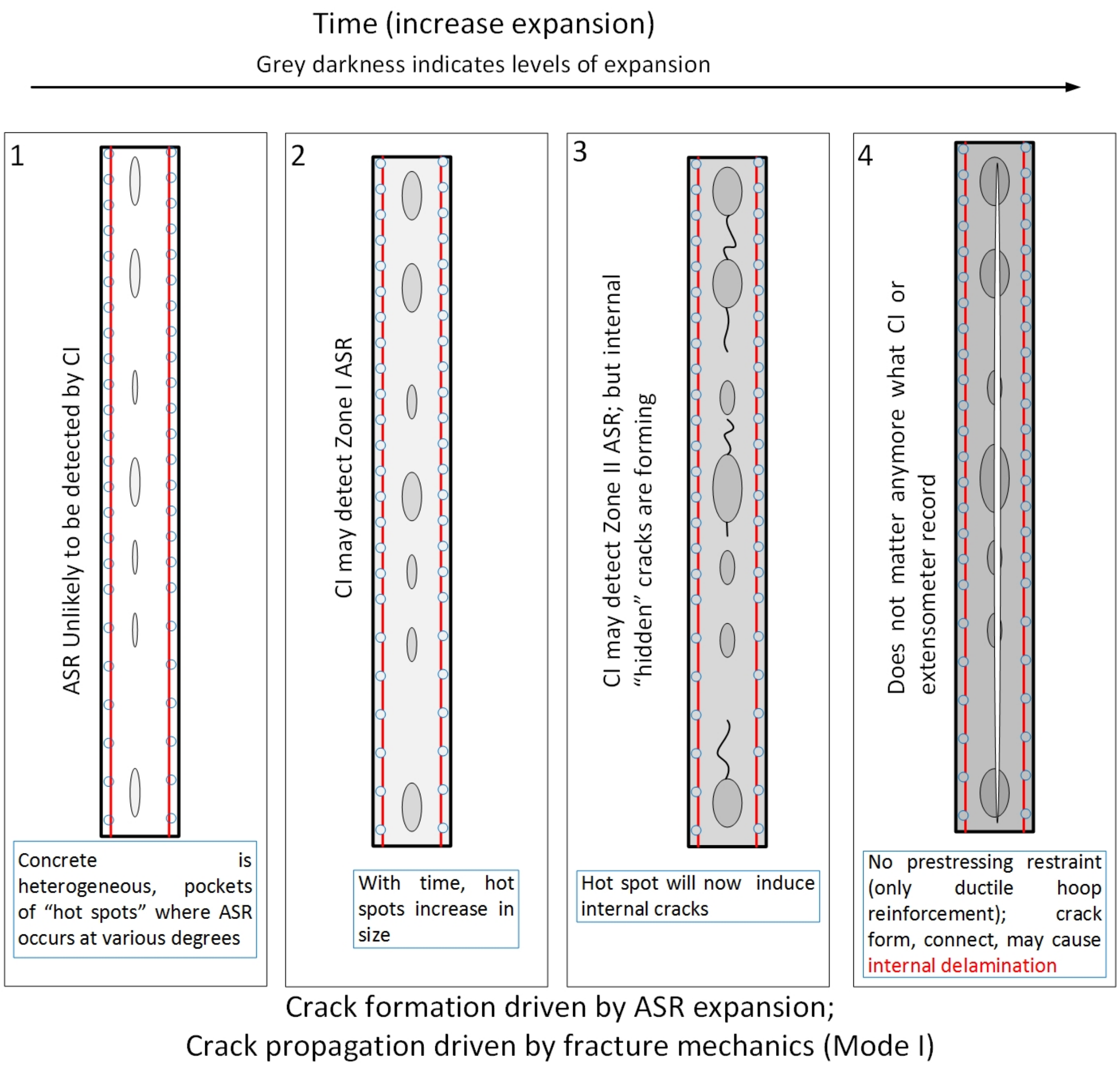


Figure 15 Very likely delamination mechanism inside Seabrook

Should there be any doubt about this, Figure 16 illustrates delamination cracks in reinforced concrete retaining walls in Switzerland. The walls, just like Seabrook (or the LSTP shear specimens) have reinforcement on the surface but not across the thickness. As a result of this, ASR caused the delamination of the walls.

Figure 16 Examples of ASR driven delamination in surface reinforced concrete retaining walls in Switzerland (Courtesy Dr. Leemann, EMPA, Switzerland)

This potential internal and hidden/sleeping crack is a very major concern.

D.7.3 In A103, SGH's witnesses criticize your testimony that the elastic modulus E is undoubtedly affected by ASR. Please comment.

SGH takes the position that there is no need to reduce the stiffness in the reinforced direction (confined), but that in the unconfined direction it may be reduced. If so, then the finite element analysis should be based on an anisotropic constitutive model, one where the elastic properties remain intact in the longitudinal and circumferential plane, but is reduced in the radial direction. This is important, because we remain concerned about the possibility of delamination.

This was not done in the finite element analysis, and this constitutes yet another concern.

Furthermore, there is a contradiction in NextEra's position. It exploits the reduction in E for the determination of the through expansion, but ignores it in the finite element analysis (though the flexural stiffness seems to be reduced).

By modeling the ASR as a temperature load, NextEra is compounding the problem because:

1. The expansion is predominantly in the radial direction, and the temperature load induces equal expansion in all three directions.
2. The absence of reduced elastic modulus in the concrete will inhibit the radial (thermal) expansion.

D.7.4 In A113, SGH's witnesses cite a set of research papers they claim to support their assumption that concrete can be expected to swell between 0.01% and 0.02%. Do their citations persuade you?

No. This is an arbitrary estimate of the possible maximum ASR expansion. The only way to make such an estimate for the future expansion is to perform an accelerated expansion test. I will note that these tests are extensively reviewed in an EPRI Report 3002013192 (Evaluation of Laboratory Tests to Detect Up-to-Date Expansion and Remaining Expansion in Concrete Structures Affected by Alkali-Silica Reaction" (Oct. 15, 2018)), Exhibit NER018.

D.8 Monitoring

D.8.1 In A187, MPR's witnesses state that monitoring intervals at Seabrook are based on "observed conditions," with a monitoring frequency that is based on CCI values, with frequency increasing as those values increase from 0 (walkdowns every 5-10 years) to below 1.0 mm (monitoring of in-plane expansion every 2.5 years) to 1.0 mm or greater (monitoring of in-plane expansion, through-thickness expansion, and volumetric expansion every 6 months). In A81, SGH's witnesses also state that in order account for future ASR growth, a "threshold factor" is used to increase the current ASR loads. SGH's witnesses characterize this approach as "conservative" because "future ASR growth is expected to slow down, since the amount of reactive material decreases over time."

I am concerned that both the monitoring program and SGH's analysis are based on a concept of linear growth in ASR, and is therefore not scientifically valid, realistic or conservative. As I have previously testified, the kinetics of the reaction follows a sigmoid curve, Figure 17. Once the latency time (time until the curve "kicks off" upward) has been reached, expansion increases rapidly. At Seabrook, in the absence of modern Petrographic DRI investigation, we have no idea where the progress of ASR lies along this curve. Hence, there is a significant risk that inspection intervals may be too long, or that given the randomness of the "hot spots" of ASR reaction, we may miss an active ASR expansion at one location.

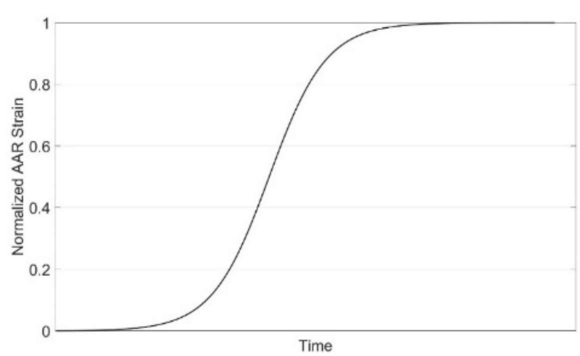


Figure 17 Typical expansion curve for ASR

Indeed in (Kojima et al, 2000) It is reported that "*The reactivity of aggregate and* *<u>sudden expansion</u>* *of the concrete were first discovered in Osaka…",* (emphasis added).

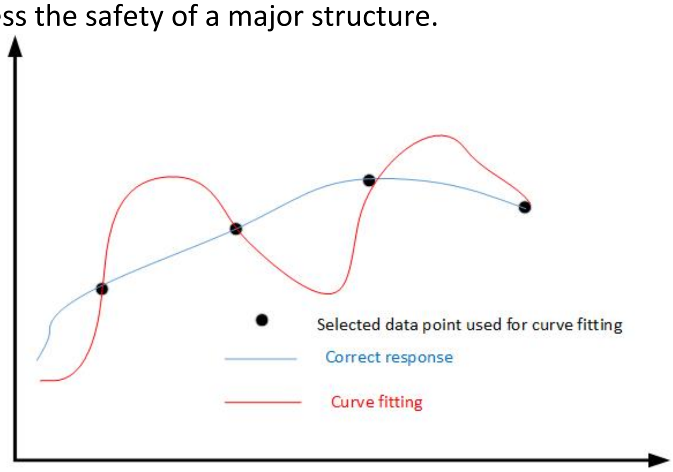
In (DaSilva and Oliveira, 2008) the authors state: "*[ASR] is a phenomenon that develops through the years, it cannot be said that the limit state rupture will not be attained in spite of the delayed reaction. Therefore, concrete elements affected by the AAR must suffer interventions with appropriate recovery techniques for each situation.*"

As these authorities recognize, ASR expansion can take off and result in substantial damage.

D.8.2 <u>In A83, SGH's witnesses state that "Strains are always monitored," and that "Strains are used as input to the model and can be monitored by CI, CCI." Please comment</u>.

This calls for the following observations:

1. I am not surprised that the model was "validated." Indeed, if one applies a load temperature corresponding to the strains measured in-situ, then it is not surprising that the measured strains will match the measured ones.
2. Having the finite element tuned to capture selected deformations is far from having a reasonably accurate model. This is analogous to "curve fitting" where we may be able to match a selected point, but not the entire curve Figure 18. This problem plagues all analyses. Hence, one tries to minimize this problem by using appropriate analysis (proper ASR modeling, proper finite element model). Only then would the results be sufficiently correct to assess the safety of a major structure.



<u>Figure 18 Problem with curve fitting</u>

Finally, it should be mentioned that nonlinear analysis does not imply that one is using an "extraordinary/uncommon" approach. Such analyses are commonly performed by companies like WJE, Weidlinger and others.

D.8.3 In A215, MPR's witnesses criticize you for suggesting that concrete should be tested for free chloride concentration. They say it is irrelevant to ASR. Please comment.

MPR's witnesses misunderstand my testimony. I did not argue that chloride would contribute to further ASR expansion. However, Seabrook is very close to several bodies of saltwater (marshes and ocean), and is extensively cracked. Hence, saline solution could easily find its way through the ASR-induced cracks, depassivating the steel rebar (according to Faraday's law), and causing corrosion (Hansen and Saouma, 1999)

D.9 Structural Evaluation Methodology

D.9.1 Please discuss your concerns about the "corroboration study" as described by MPR's witnesses in A95.

As described by MPR's witnesses, the corroboration study "focuses on a correlation developed during the LSTP that is used by NextEra to estimate through-thickness expansion at Seabrook before an extensometer is installed. According to MPR, it is "an approach for obtaining in-plant data to evaluate how expansion at the plant aligns with observed expansion of the LSTP specimens." In A176, MPR states that the corroboration study will occur several years after installation of the extensometers "to allow time for through-thickness expansion to occur."

MPR's testimony concerns me because it fails to acknowledge how much the corroboration study depends on approximating quantitative values related to ASR. For instance, it necessitates:

1. Determination of the Elastic modulus of the concrete cast during construction based on the ACI-318 empirical formula.
2. Determination of current elastic modulus (presumably using ASTM tests and not using the empirical equation).
3. Normalize the current elastic modulus with the empirically determined one.
4. Correlate the normalized elastic modulus with in-situ through thickness expansion.

Each one of these determinations carries substantial uncertainties. With regard to the first one, estimating elastic modulus from 28 days' compressive strength (determined during construction) carries numerous uncertainties as investigated by (Geyskens and Kiureghian, 1998)

As to the correlation between normalized elastic modulus and through thickness expansion, it is based on few test data at the FSEL and do have an inherent variability.

In summary, there are two major uncertainties as indeed reported by NextEra:

1. NextEra compared the values of the elastic modulus determined from the empirical equation with the exact one measured in the laboratory. This comparison is necessary because NextEra plans to estimate the elastic modulus of the concrete cast over 30 years

ago using the empirical equation (as it is most unlikely that back then tests for elastic modulus were performed). The comparison is shown in Figure 19. An analysis of the data shows that the lower and upper bound correspond to 80% and 120% respectively (as expec [redacted]

Figure 19 Comparison between ACI empirical equation and exact value for E [PROPRIETARY INFORMATION]

2. Another uncertainty is associated with the relationship between the through thicknesses and normalized elastic modulus. The reported data and fitted curve by NextEra are shown in Figure 20 (Exhibit NRC012, MPR-4153, Rev. 2, "Seabrook Station - Approach for Determining Through Thickness Expansion from Alkali-Silica Reaction," (July 2016)) (PROPRIETARY)

[redacted]

Figure 20 Comparison between through thickness expansion and normalized elastic modulus (Exhibit NRC012, MPR-4153, Rev. 2, "Seabrook Station - Approach for Determining Through Thickness Expansion from Alkali-Silica Reaction," (July 2016)) [PROPRIETARY INFORMATION]

I analyzed the impact of both uncertainties, Figure 19 and Figure 20 to determine what would be the possible margin of error in NextEra's curve (shown in Figure 21). The result is shown in Figure 22. The approach is described next:

[redacted]

Figure 21 NextEra's "adjusted" correlation curve (note the absence of error bars). [PROPRIETARY INFORMATION]

1. Figure 22 uses data obtained from laboratory cast cores and are in the range of [redacted] (Figure 194).
2. Since this curve is based on concrete cast over 30 years ago in the field, the range was (arbitrarily but reasonably) broadened to [redacted].
3. Data from Figure 20 are used to capture the variability between normalized stiffness and laboratory measured through thickness.
4. Results are plotted with the horizontal x-axis corresponding to the normalized elastic modulus E (as customarily done, the x-axis corresponds to the "input"), and the y (vertical axis) corresponding to the (output) through thickness. Note that NextEra presented its curve (Figure 21) in the reverse direction.
5. Select a normalized value of the elastic modulus to be 0.4.
6. Due to uncertainties in E, the minimum and maximum possible values would be [redacted]
7. In Figure 22 we go up the y axis to determine the through thickness, and note that it would be somewhere between [redacted]% and [redacted]%.

Clearly, this margin of error is simply unacceptable given the sensitivity of the structure and the nonlinear rate of expansion.

Hence, there would be a <u>very low confidence</u> level associated with what NextEra perceives to be the total expansion. This is critical because it is precisely this expansion that is inputted to the SEM to ultimately assess the safety of Seabrook.

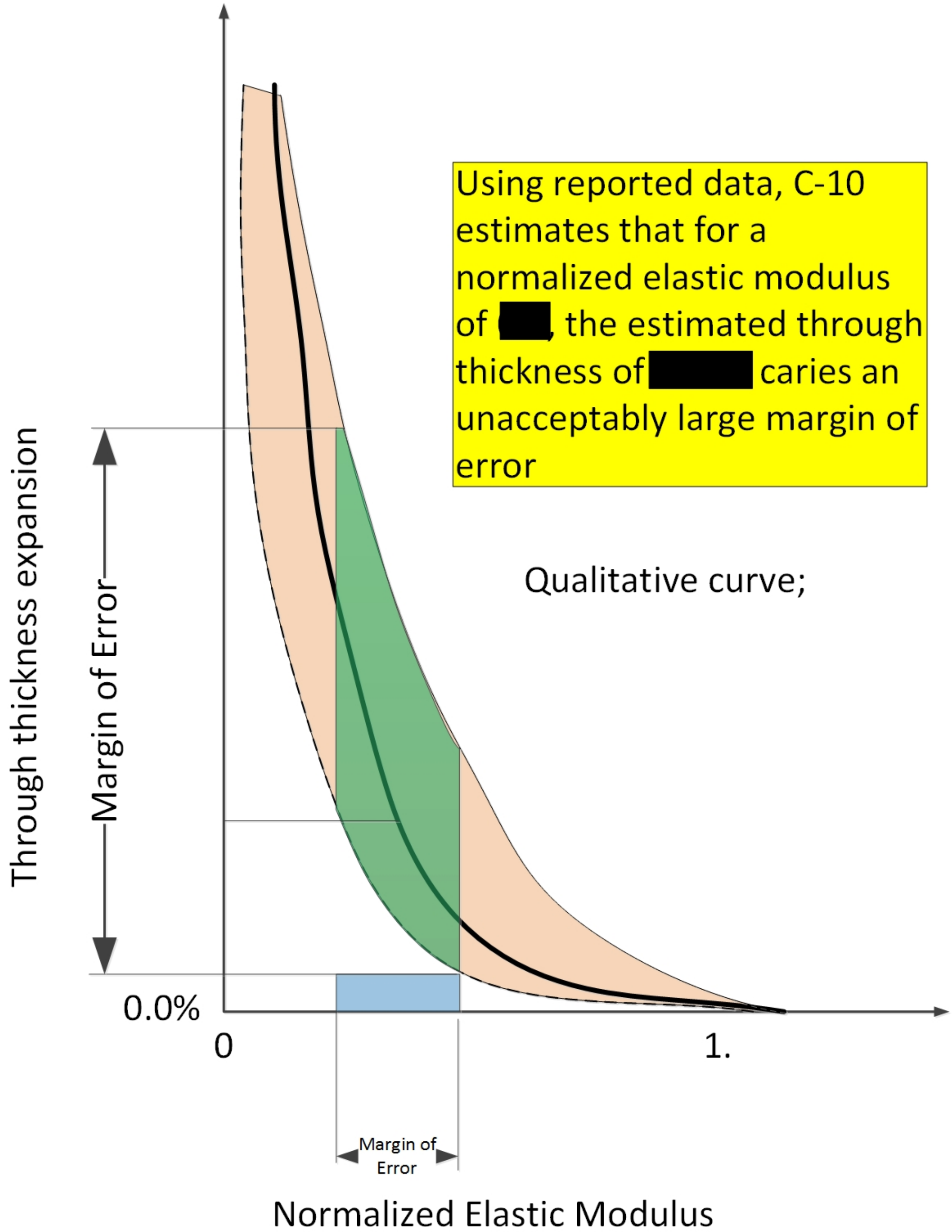


Figure 22 Uncertainty in the determination of the Elastic modulus from the compressive strength as proposed by NextEra [~~PROPRIETARY INFORMATION~~]

Finally, it should be emphasized that this entire procedure would not have been possible without the validation of the methodology through the LSTP. Hence, once again the tight coupling between LSTP, SMP, and SE is proven without any doubt.

D.9.2 In A76, SGH's witnesses describe the "ASR load inputs" to the finite element modeling used to evaluate total demand on the CEB structure. According to SGH:

The ASR load inputs to these models are: (1) the internal in-plane ASR expansion of reinforced structural members, and (2) the pressure due to ASR expansion of the concrete fill. The internal ASR expansion is determined via the field-measured CI expansion strain; CI is measured in each of the in-plane orthogonal directions. CI represents an equivalent ASR strain. FEM codes do not provide direct inputs for ASR expansion, but thermal expansion can be used as a proxy, and therefore ASR strain is simulated by applying an equivalent load to the concrete only. External ASR pressure may also be exerted by expansion of the concrete fill. . . The ASR effects of concrete backfill are simulated by applying pressure on adjacent embedded members. . .

Please comment.

I am concerned that the approach to modeling ASR described by SGH is rudimentary and erroneous, and it grossly oversimplifies the problem. SGH disregards the universal consensus that:

1. ASR is a function of temperature (and there is a temperature gradient across the wall of the CEB).
2. ASR is a function of the relative humidity (below 80% no expansion). There is a RH gradient across the wall.
3. ASR is a volumetric expansion and is anisotropic. As confirmed by the LSTP, there is reduced expansion along the reinforcement (in plane for the CEB), and increased expansion across the thickness (direction along which there is no reinforcement).
4. When confinement reaches approximately 8MPa, there is no more expansion in that direction.
5. ASR causes a reduction of material properties as it expands.

None of the above can be captured by SGH's model.

SGH may want to consult the recent EPRI (2017) report (to which Saouma contributed an important chapter on modeling), Please note that the price of this report being $25,000 for non EPRI member, I was only given a draft copy. Furthermore, SGH should also consult more closely Gocevski on ASR (Exhibit NER038). It will attest to the inherent complexity of modeling ASR that is certainly not addressed in the SGH analysis.

D.9.3 In A75, SGH states that the Finite Element Model uses the concrete properties specified in the original construction documents for Seabrook, under the proviso that the structure remains within the Expansion Monitoring Limits in the SMP. Do you agree with this approach?

No. The linear elastic approach is fundamentally inadequate for such an investigation. Cracked section properties are (mostly) the result of ASR. Hence, a correct analysis should start with a "virgin" concrete, simulate the ASR, and in doing so capture numerically the ensuing cracking and the capacity to resist the demand. It is incorrect to start with cracked properties.

Case in point, the simplified analysis method (ACI 318-71, Eq.9.4) relies on the analyst to determine the so-called reduced moment of inertia (to account for the cracking). But this reduction is a function of the applied demand (moment Me defined as maximum moment in member). However, when the analysis starts, this value is unknown, and must be estimated. This makes the problem non-linear.

Most importantly, the ACI code for the reduced moment of inertia assumes that cracking is due to service loads, resulting in the classical pattern of flexural or flexural shear cracks. Now we have much more cracking, cracking caused by ASR. Hence, the moment of inertia may be grossly overestimated, and as a result one would be under-estimating the deflections, and more importantly underestimating the structural cracks which may provide pathways to leaks.

D.9.4 In A29, the NRC Staff also states that:

> [A]ny further increase in frequency due to ASR effects is expected to also result in a decrease in seismic demand. Also, uncertainties in the impact of ASR on stiffness are expected to be accommodated by the 10% peak broadening of the response spectra.

Please comment on this assertion.

In making this statement, the NRC Staff acknowledges that NextEra's prediction regarding the effect of ASR on stiffness is uncertain. However, the 10% estimate is unsupported and therefore speculative. The NRC Staff does not say, nor can it be determined, whether 10% peak broadening the response spectra is sufficient or not.

Hence, I remain critical of the assumption in the LSTP that the natural frequency of the damaged CEB is the same as the one in the original condition. A more refined study should have been conducted (indeed the adopted "Stick elements") are as simple as can be in this context.

E Final Remarks

E.1 Please summarize the key conclusions of your rebuttal testimony.

1. NextEra's testimony reflects a narrow code-based engineering approach rather than a combination of scientific and engineering approaches as is required for a problem with

the complexity of ASR in the concrete enclosure of a nuclear reactor. The contrast can be seen by comparing the Seabrook ASR project to Hydro-Quebec's investigation of ASR at the Gentilly-2 nuclear plant, a much more sophisticated and effective investigation.

2. Through their testimony, NextEra's and the NRC Staff's witnesses have demonstrated a lack of sufficient expertise in the field of ASR assessment and analysis. This lack of expertise was compounded by a failure to obtain independent peer review of NextEra's work.
3. NextEra's proposed code-based-engineering approach may comply with the 1971 ACI code. However, the corresponding margin of error has not been quantified and is likely to be unacceptable. This makes it too risky to be adopted.
4. ACI 318-71, is not an adequate tool because it stipulates linear elastic analysis rather than inelastic (i.e., nonlinear) analysis. Use of nonlinear analysis is common and widely performed in the 21st century for complex structures such as Seabrook (as in the *post-mortem* investigation of Crystal-River). It should have been employed at Seabrook.
5. The finite element simulation used by NextEra is very rudimentary, completely inappropriate modeling of the ASR and the possible in-plane degradation.NextEra has failed to recognize some of the key characteristics of ASR, namely the driving force of relative humdity, the relationship between the characteristics of aggregates to both the nature of cracks and the timing of their development, the kinetics of the ASR reaction, the degradation of concrete mechanical properties in the SEM, and the lack of uniformity in the location and progression of ASR. NextEra also fails to acknowledge that the progression of ASR over time follows a sigmoid curve and is not linear. NextEra's failure to account for all these factors plays a significant role in undermining the reliability of its assessment of ASR.
6. I do not give much credence to the shear tests for the purpose of assessing the impact of ASR and the ultimate strength of the beam. Those results could have been easily anticipated and confirmed by proper finite element studies (i.e. they were un-necessary in my opinion). On the other hand, a by-product was the development of the inspection methodology.
7. The environmental conditions under which the CI and through crack extension were measured in the laboratory do not correspond to the conditions at the Seabrook Plant. As a result, the extent of internal expansion will most certainly be misleading. Furthermore, NextEra failed to recognize the impact of the reinforcement close to the surface of the wall that would inhibit crack opening.
8. Due to the confinement, the expansion will be radial, hence the cracking will be internal and propagate vertically (along the lines of compression). Furthermore, it will seldom daylight to be captured by CI. Hence, the walls of the CEB could very well delaminate internally, and this delamination will either not be captured by the instrumentation, or not captured in a timely way.

9. The ten-year effort to understand ASR at Seabrook and establish a program to adequately monitor ASR's progression over the next 30 years (including the remainder of Seabrook's current license term and a 20-year renewal term) has fallen far short of providing a reasonable assurance that NRC seismic design requirements are satisfied and that the public will be protected in the event of an earthquake. Yet, some progress has been made, especially in the program to install extensometers for more accurate monitoring.
10. In my expert opinion, NextEra should return to the drawing board, applying greater expertise, collecting more meaningful data, and using more appropriate and commensurate scientific and engineering approaches. Some of the work that has been done will still be useful and should be expanded on, such as the use of extensometers. But overall, the investigation should take a new approach that is more scientific, rigorous and sophisticated and subject it to a panel of independent expert reviewers in various related disciplines.

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