UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

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

Hearing Exhibit

Exhibit Number: INT027

Exhibit Title: PRE-FILED OPENING 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

REDACTED VERSION FILED JUNE 26, 2019

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SUBMITTED ON BEHALF OF C-10 RESEARCH AND EDUCATION FUND

June 10, 2019

REDACTED VERSION JUNE 26, 2019

AVAILABLE TO PUBLIC

Table of Contents

List of Figures

EXHIBIT INT001

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

EXHIBIT INT001

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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. And I am *Professeur des Universités* in France.

A.2 Please identify this document.

This is my testimony regarding my scientific evaluation of NextEra's Aging Management Program for Alkali-Silica Reaction at the Seabrook nuclear power plant. My written testimony is submitted in two versions: **EXHIBIT INT001** is my complete testimony**,** and includes some proprietary information. I am also submitting **EXHIBIT INT002,** which contains the introductory section and a summary of my conclusions. I also plan to submit a redacted version of Exhibit 1 as soon as possible.

A.3 Please describe your professional qualifications to give this testimony.

I am a leading international expert in the field of Alkali-Aggregate Reaction (AAR), which is also known as Alkali Silica Reaction (ASR). I am not aware of any other researcher (other than one in France) who has conducted the same breadth and depth of research on ASR: theoretical, numerical (deterministic/probabilistic, static/dynamic), experimental (material and structural). I have developed what is probably the most widely referenced and copied model for ASR, the "Saouma Model." The Saouma Model is used by the Idaho National Laboratory in the Abaqus, Vector3, and Grizzly/Moose computer programs. It is also used by HydroQuebec for dam analysis. And it is used as well in China, Switzerland, and Canada.

I have conducted research for numerous government agencies, including the U.S. Nuclear Regulatory Commission (NRC), the U.S. Army Corps of Engineers, the U.S. Department of the Interior's Bureau of Reclamation, the U.S. Department of Energy's Oak Ridge National Laboratory, the National Science Foundation, the Tokyo Electric Power Company (TEPCO), and the Swiss Federal Office for Water Management (dam safety). My research has encompassed material and structural testing, theoretical and computational modeling, fracture mechanics, risk-based numerical assessment of bridges, nuclear containment structures and dams, chloride diffusion, and experimental dynamics. I have written about 100 peer-reviewed articles on these topics, including approximately 30 articles on ASR and the related topics of chloride diffusion, seismic analysis and stochastic analysis. I have also written a book on numerical modeling of ASR, *Numerical Modeling of Alkali Aggregate Reaction* (CRC Press 2013).

I have been a consultant (providing expertise in fracture mechanics) to Performance Improvement International (PII) investigating the root cause of Crystal River nuclear containment delamination.

In addition, I serve or have served on numerous scientific organizations, committees, and panels, including current chair of a RILEM (French acronym of *International Meeting of Laboratories and Experts of Materials, Construction Systems and Structures*) committee on Diagnosis and Prognosis of ASR affected Structures, RILEM TC 259-ISR. And I am the past president of the International Association of Fracture Mechanics for Concrete and Concrete Structures.

A copy of my curriculum vitae is attached to my testimony as **EXHIBIT INT003.**

A.4 Have you done any research or writing specifically related to ASR at Seabrook?

In 2014, I co-authored a journal article regarding aging management of ASR at Seabrook. The article presented a scholarly assessment of the gap between the reported methodology and the state-of-the-art, based on the limited amount of information that was publicly available at the time (Saouma, V. E., & Hariri-Ardebili, M. A., 2014).

In addition, in 2014, the NRC awarded me a three-year $703,000 contract to provide support for a project entitled "Experimental and Numerical Investigation of Alkali Silica Reaction in Nuclear Reactors." A copy of the grant award is attached as **EXHIBIT INT004.** As stated at page 4 of the grant award, the impetus for my proposed research stemmed from "the apparent challenge confronting the NRC in assessing safety issues pertaining to the Seabrook nuclear power plant which suffers from Alkali Silica Reaction (ASR), and in particular NRC's request that the licensee determines the long term safety of the plant within the framework of [Seabrook Alkali Silica Reaction Issue Technical Team Charter (July 9, 2012)] ML121250588 (2012)."

My research ended in December 2017 when I submitted a four-volume final report. A copy of the Final Summary Report is attached as **EXHIBIT INT005.**

To date, to the best of my knowledge, our study for the NRC is the most comprehensive on the effect of ASR on the shear strength of concrete. Sixteen large specimens were carefully prepared and tested using a unique apparatus designed for shear testing. It was determined that a 0.6% expansion reduces strength by 20%. We found that ASR of a relatively low 0.3% reduced the resilience of an NCVS subjected to seismic excitation by approximately 20%. We also successfully demonstrated the applicability of a modern probabilistic based static/dynamic nonlinear methodology for evaluating ASR.

In addition, in 2018 I was retained by the C-10 Research and Education Foundation (C-10) to evaluate work done by NextEra, NextEra's consultants, and the NRC technical staff regarding the presence of ASR in concrete at the Seabrook nuclear power plant; and the effect of ASR on the integrity of the concrete, including the containment. In the course of my evaluation, I reviewed both public and proprietary documents regarding NextEra's investigations. I also applied the

insights of my work under the NRC contract described above. C-10 submitted my declaration and report to the NRC Commissioners in support of an emergency petition to further address ASR at Seabrook before re-licensing the reactor. A copy of my declaration is attached as **EXHIBIT INT006.** A copy of my expert report is attached as **EXHIBIT INT007 (PROPRIETARY).** A publicly available summary of my expert report is attached as **EXHIBIT INT008.** And a copy of the Reply Declaration I submitted in support of my expert report is attached as **EXHIBIT INT009.**

A.5 What documents have you reviewed in preparing your testimony?

I have reviewed NextEra's license amendment request (LAR), *Seabrook, License Amendment Request 16-03 - Revise Current Licensing Basis to Adopt a Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali- Silica Reaction* dated August 1, 2016 ("Letter SBK-L-16071") (ML16216A240) **(EXHIBIT INT010)**, including its subsequent revisions.

The major elements of the LAR and the consultant reports that I have evaluated are as follows:

NextEra Energy's Evaluation of the Proposed Change Including Attachment 1 Markup of UFSAR Pages (Proprietary) (Enclosure 1 to Letter SBK-L-16071) **(EXHIBIT INT011) (Proprietary)**;

MPR-4288, Rev. 0, *"Seabrook Station: Impact of Alkali-Silica Reaction on Structural Design Evaluations* (July 2016) (Non-proprietary version) (ML16216A241) (Enclosure 2 to Letter SBK-L-16071) (**EXHIBIT INT012)**;

SG&H Report 160268-R-01, Rev. 0, *Development of ASR Load Factors for Seismic Category I Structures (Including Containment) at Seabrook Station, Seabrook, NH* (July 2016) (ML16216A243) (Enclosure 4 to Letter SBK-L-16071) (**EXHIBIT INT013)**;

MPR-4288, Rev. 0, *"Seabrook Station: Impact of Alkali-Silica Reaction on Structural Design Evaluations* (July 2016) (Proprietary Version) (Enclosure 5 to Letter SBK-L-16071) **(EXHIBIT INT014) (Proprietary)**;

Simpson Gumpertz & Heger, Inc., *Evaluation and Design Confirmation of As-Deformed CEB, 150252-CA-02"* Revision 0, July 2016. (ML16279A049) (2016) (Enclosure 2 to Letter SBK-L-16153, re: Seabrook Station (Sept. 30, 2016)) (**EXHIBIT INT015).**

Revised Seabrook Station License Renewal Application Updated Final Safety Analysis Report Sections A.2.1.31 for Structures Monitoring, A.2.1.31A for Alkali-Silica Reaction and A.2.1.3b for Building Deformation (Enclosure 1 to Letter SBK-L-18072 re: Seabrook Station Revised

Structures Monitoring Aging Management Program (May 18, 2018) ("Letter SBK-L-18072")) **(EXHIBIT INT016)**;

Revised Seabrook Station License Renewal Application Appendix B Sections B.2.1.31 for Structures Monitoring, B.2.1.31A for Alkali-Silica Reaction and B.2.1.3b for Building Deformation (Enclosure 2 to Letter SBK-18072), **(EXHIBIT INT017)**;

MPR-4153, Revision 3, *Seabrook Station-Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction* (Sept. 2017) (Non-proprietary version) (ML16279A050) (Enclosure 4 to Letter SBK-18072) (**EXHIBIT INT018)**;

MPR-4273, Rev. 1, *Seabrook Station - Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction* (July 2016) (Non-proprietary version) (ML18141A785) (Enclosure 5 to Letter SBK-18072) (**EXHIBIT INT019)**;

MPR-4153, Revision 3, *Seabrook Station-Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction* (Sept. 2017) (Proprietary version) (Enclosure 6 to Letter SBK-18072) **(EXHIBIT INT020) (Proprietary)**;

MPR-4273, Rev. 1, *Seabrook Station - Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction* (March 2018) (Proprietary version) (Enclosure 7 to Letter SBK-18072) **(EXHIBIT INT021) (Proprietary)**;

Simpson Gumpertz & Heger Document No. 170444-MD-01, Rev. 1, "*Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction," for Seabrook Station* (Enclosure 3 to Letter SBK-L-18074, re: Seabrook Station, Response to Request for Additional Information Regarding License Amendment Request 1603 (June 7, 2018) ("Letter SBK-L-18074")) (**EXHIBIT INT022)**

Simpson Gumpertz & Heger Document No. 170444-L-003 Rev. 1, *Response to RAI-D8-Attachment 1 Example Calculation of Rebar Stress For a Section Subjected to Combined Effect of External Axial Moment and Internal ASR* (Enclosure 4 to Letter SBK-L-18074) (**EXHIBIT INT023)**;

In addition, I have reviewed the publicly available and proprietary versions of NRC's Safety Evaluation Related to Amendment No. 159 to Facility Operating License No. NPF-86 (March 11, 2019) (publicly available version at ML18204A291) (**EXHIBITS INT024** (public) and **INT025 Proprietary).**

I have reviewed applicable government and industry standards.

I have also reviewed the Licensing Board decision admitting C-10's contentions, LBP-17-07, 86 N.R.C. 59 (2017).

Finally, I have reviewed a large body of research reports and academic literature regarding the phenomenon of ASR.

A.6 Please describe the purpose of your testimony.

The purpose of my testimony is to provide technical support for C-10's assertion that the large-scale test program, 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. My testimony will also address C-10's particular concerns regarding the insufficiency of crack width indexing and extensometer deployment for determining the presence of ASR, NextEra's misconception of the effects of ASR within a reinforced concrete structure, the need for continuous petrographic sampling and analysis, and the unacceptability of the proposed length of intervals between inspections.

A.7 Why are you providing this testimony?

I am providing my testimony to C-10 *pro bono,* because I am very concerned, both as a scientist and a citizen, about the inadequacy of the work that has been done on ASR at Seabrook. To address a problem as complex and potentially dangerous as ASR, it is essential to avail oneself of the best possible information and expertise. Therefore, it disturbs me that neither NextEra nor the NRC has sought to apply the current state of knowledge regarding ASR or to obtain independent review of their work. Instead, they have offered assurances of safety to the public that are based on simplistic analyses, erroneous assumptions, and data that are not representative of conditions at Seabrook. These analyses and data were far from adequate to give the NRC technical ground to continue to operate Seabrook during its current license term or to re-license Seabrook for another 30 years (*i.e.*, until 2050).

B Background: ASR

B.1 Please describe the phenomenon of Alkali Silica Reaction (ASR)

Alkali Silica Reaction is a chemical reaction in concrete caused by a Ph imbalance. Cement and some aggregates are responsible for the alkalinity, and the silica inside aggregates provides acidity. Under conditions of high relative humidity (at least 80%), ASR results in the formation of a viscous gel (with calcium playing a major role in the viscosity of the gel). The expanding concrete first fills up voids, and then causes the concrete to expand. The kinetics of the reaction (that is the rate of expansion) is a function of time, temperature and concrete relative humidity. ASR is almost never homogeneously spread over a large structure, because reactive concrete tends to occur in "pockets" where silica-rich aggregates may have been used. Heterogeneous distribution

of ASR (as is the case of Seabrook) is more problematic than homogeneous distribution, because it will cause gradients of expansion (think of the Tower of Pisa with unequal settlement).

ASR progress depends very much on the geological nature of the aggregate and sand. In some cases, we have an early-expansion (such as rhyolitic aggregate), and in others a late-expansion (such as granite). Furthermore, sand will result in a rapid expansion, and aggregates will cause a slower, but larger, future expansion. Hence, it is nearly impossible to duplicate a reactive concrete unless one uses exactly the same concrete mix and ingredients.

If unimpeded, ASR expansion is volumetric and isotropic (*i.e.,* the same amount of expansion occurs in three directions or "planes"). However, confinement of the concrete will inhibit ASR expansion in those directions and reorient it along the direction of least confinement. Confinement in Seabrook and other nuclear plants is lateral due to geometry, and vertical due to geometry and weight of the reactor; hence expansion will be mostly out of plane, that is radial.

The ultimate effects of ASR include both expansion and degradation of the concrete mechanical properties. This combination of expansion and degradation affects tensile and shear strengths along with elastic modulus. Tensile strength will control the formation of (undesirable) cracking, and the elastic modulus degradation will result in larger deformation and potential cracking. The decrease in shear strength can compromise the integrity of a containment during an earthquake.

Many tests have shown an increase in structural shear strength in reinforced concrete beams (through the so-called prestressing effect) because of ASR. This is not to be confused with the inherent shear strength of plain concrete material, which is not strengthened by ASR but rather is degraded.

B.2 <u>What legal or industry standards are applicable to ASR?</u>

ASR is a relatively new, complex and potentially dangerous problem. I am aware of no regulations or industry standards that have been developed to specifically address the presence of ASR and its implication on serviceability and strength.

The federal highway administration (FHWA) has published a number of reports (written by leading experts) addressing this problem and providing a road map on how to deal with ASR using modern tools. For example, see Fournier, B., Berube, M. A., Folliard, K. J., & Thomas, M. (2010).

The U.S. government has invested significant resources into research on ASR, including my contract, a grant of approximately $7 million to the National Institute of Standards and Technology (NIST), and U.S. Department of Energy (DOE) research through the Oak Ridge National Laboratory. Given this investment, I would have thought that the NRC and DOE would develop similar guidelines for the nuclear industry as the FHWA. This did not happen, and for all practical purposes it was effectively left to NextEra to write their own guidelines through their License Amendment Request.

B.3 Can we treat safety assessment of an existing structure suffering from ASR the same way we designed it?

In evaluating the degree to which ASR threatens compliance with NRC safety standards, it is important to bear in mind that analytical considerations related to the design of new structures are very different from the ones relating to the safety of existing structures. Analysis for design of new structures starts by amplifying the load by say 40 or 50%, and the response up to failure is assumed to be linear (this is indeed code-driven). In analyzing the safety of existing safety structures, one has to determine the exact nonlinear response beyond the elastic limit and -- most importantly -- determine the corresponding deformation that would be under-estimated in the former case. Only the latter approach can truly capture the impact of damage caused by ASR without over-simplification. See Figure 1. A simplified linear elastic analysis (used in design of new structures) will under-estimate the displacements and cannot capture either the failure load or the deformation. Safety assessment can only be performed through a nonlinear analysis.

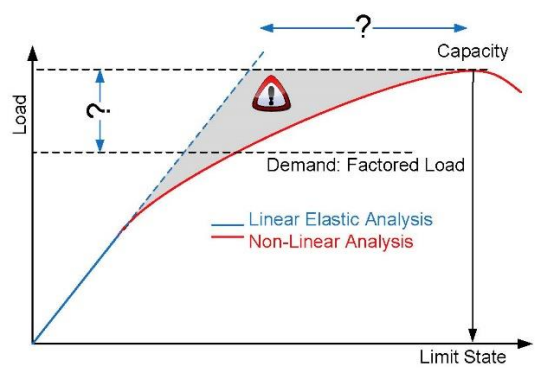


Figure 1 Design vs Analysis

C Discussion of Expert Opinion

C.1 Do you continue to hold the same opinions you expressed in your February 2019 expert report?

Yes. My conclusions about the adequacy of NextEra's investigations are reflected in my report entitled Concerns Regarding the Structural Evaluation of Seabrook Nuclear Power Plant (Feb. 12, 2019) (Exh. INT007). As I stated there, both the FSEL testing program and the finite element analysis used by NextEra's consultant SG&H (with important input from the laboratory tests) are "substandard and inadequate to support any conclusion that the ability of the Seabrook containment to withstand a design basis earthquake has not been unduly compromised by the presence of ASR." *Id.* page 3. To summarize, insufficient attention has been given to the unique and complex nature of ASR. Therefore, based on my expertise and the published state of the art of ASR and basic principles of structural engineering (design vs safety assessment; linear vs

nonlinear analysis), I concluded that the quality of the presented results is not sufficiently reliable to support their stated purpose of confirming regulatory compliance for the next 30 years.

As discussed in my expert report, the manner in which NextEra's consultants have analyzed the impact of ASR on Seabrook is seriously deficient in five major respects.

- First, the concrete used in the FSEL testing program was not representative of the concrete at Seabrook.
- Second, the specimen (scaled) dimensions, loads and boundary conditions are not representative of Seabrook.
- Third, NextEra's failure to address reported load displacement and cracking patterns; and
- Fourth, NextEra relies on incorrect assumptions about ASR, including confusing material strength with structural strength and assuming that adding a "design basis load" to the Seabrook safety analysis can account for ASR.

By themselves, these errors, which were incorporated into NextEra's finite element analysis, had the effect of rendering that analysis completely unreliable to support any conclusions about the safety of the Seabrook plant under earthquake conditions.

And the errors adversely affected the adequacy of parameters used in the monitoring program. Both the monitoring program for ASR progression and the monitoring program for structural deformation depend on FSEL test results. And both programs are seriously deficient because of that dependence.

In addition, the problems with NextEra's safety assessment and monitoring programs were compounded by the fact that NextEra and its consultants applied an analytical method to the FSEL data that was extremely simplistic and contains numerous significant flaws (ASR modeling and seismic analysis, among others.) By feeding erroneous and unreliable data into an analytical model that was already inadequate to address the complexity of ASR at Seabrook, NextEra compounded the problem and made it even worse.

In considering this issue, it is important to recognize that testing and analysis (any analysis) is a very tightly coupled process where the latter depends greatly on the reliability of the former. Hence, the results of any finite element analysis whose cracking/failure/safety criteria depend on erroneous experimental data will consequentially be flawed.

This is what happened. First, NextEra relied on test results to conclude that Seabrook is currently safe to operate. Second, NextEra gauged the nature and degree of monitoring required based on the level of safety assurance it had obtained from the results of the flawed testing and analysis of the data. Third, NextEra based its acceptance criteria for determining the safety of Seabrook's operation and its parameters for monitoring ASR on the FSEL test results and SG&H analysis. These criteria were approved by the NRC Staff.

I also have an overarching concern about the absence of any credible peer review of NextEra's work. NextEra relied on consultants with standard engineering experience, and did not seek review by independent ASR experts. The NRC Staff ultimately accepted scientifically unproven assertions. Given that this is the very first occurrence of ASR in a nuclear containment vessel, that the NRC has funded at least two major projects on ASR, both NextEra and the NRC should have ensured that their work would receive independent review by qualified experts in the field.

It is important to note that my conclusions are relevant not just to the continued operation of Seabrook out to the end of its current license term in March 2030, but also for Seabrook's renewed operating license term – which does not expire until 2050. This is because the license amendment will become part of NextEra's Aging Management Plan for the license renewal term. Under the renewed license that has adopted the terms of the LAR, Seabrook may operate for a very long time based on (a) the LAR's unjustified determinations of safe operation and (b) a monitoring plan whose parameters were devised to confirm those faulty findings and thus are inadequate to assess the true extent and progression of ASR at Seabrook. Finally, should there be other instances of containment structures suffering from ASR, its operator will then perpetuate the flawed process endorsed by the NRC.

C.2 Representativeness of Tests

As a preamble, NextEra's LAR (**Exh. INT010)** stated: *The large-scale test programs included testing of specimens that reflected the characteristics of ASR-affected structures at Seabrook Station.* We will show that this is incorrect.

C.2.1 Concrete was not representative

The entire LAR ultimately hinges on the FSEL test program in one way or another. Yet, regretfully, the FSEL test program was not representative of Seabrook conditions. The FSEL testing program did not even meet NextEra's and NRC's own specifications for representativeness of samples with respect to ASR levels or mechanical properties. NextEra established the following specifications, for example:

> *Application of the results of the FSEL test programs requires that the test specimens be representative of reinforced concrete at Seabrook Station, and that expansion behavior of concrete at the plant be similar to that observed in the test specimens. Test specimen design addressed representativeness of the test specimens, and promoted expansion behavior consistent with the plant (e.g., use of two-dimensional reinforcement mats).*

MPR-4273, Revision 1, p. vi **(Exh. INT019)**

Furthermore, NextEra sought to establish the "*"[p]resence of ASR to an extent that is consistent with levels currently observed at Seabrook Station and at levels that could be observed in the future.*"

Id., p. 2-7.

Similarly, the NRC Staff's Safety Evaluation states that:

> *Steps [were] taken to make the MPR/FSEL LSTP as representative of Seabrook structures as possible. These included large specimen size test designs in accordance with the design basis of Seabrook and the concrete industry as a whole, reinforcement configurations and concrete mix designs that reflect Seabrook structures, and ASR levels comparable to that currently at Seabrook, as well as ASR levels that bound what could reasonably be expected in the future.*

NRC Safety Evaluation, p. 24 **(Exh. INT024).**

And Dr. Oguzhan Bayrak of FSEL stated that the concrete mixture tested at FSEL would be "*sufficiently reactive to obtain the necessary data in a timely manner"* and would *"develop mechanical properties that are representative of Seabrook structures* (Bayrak, 2012) **(EXHIBIT INT026).**

Contrary to these specifications, the concrete used in the FSEL tests was not representative of the concrete at Seabrook because FSEL and NextEra did not state that all the aggregates and all the sand came from the same quarry as Seabrook or otherwise establish that all of the sand and aggregates used in the tests were identical to what was used in Seabrook. This is an extremely important failure, because the behavior of concrete is so sensitive to variables. Different dosage (or minor additives), or different sources of aggregates, sand and cement will result in drastically different concrete. Bread provides a good metaphor. Bread is made out of flour, yeast, water, salt. Yet different dosage will result in vastly different type of breads. Likewise, different "curing" conditions will cause the yeast to result in fluffy (ciabatta) or dense (French) bread. Whole Foods in the US has been unable to replicate the French baguette, simply because we do not have the same type of flour in the US as in France. Likewise, concrete is made up of cement, sand, aggregate and water. In other words, it is not sufficient to have some expansion in your test to claim that Seabrook will behave similarly.

It is also problematic that FSEL failed to perform the accelerated expansion tests of Seabrook and FSEL concrete cores. Accelerated expansion tests would have allowed a comparison to determine the extent to which the Seabrook concrete and the tested concrete differed.

As a result of FSEL's failure to use identical concrete in its testing program, and its failure to conduct accelerated expansion tests, it is impossible to predict with any confidence the maximum expansion at Seabrook. Essentially, that figure is completely unknown. This is a significant problem that could have been easily avoided.

Likewise, the cement tested at FSEL was not reported to have the same alkali content as the one used in Seabrook. On the other hand, it is reported that "highly reactive fine aggregate (*i.e.,* sand) . . . was used and "accelerated development of ASR." (MPR-4273 Rev. 1, Section 3.1.1 **(Exh. INT019).**

In that spirit, one would reasonably assume that the concrete was doped **[**with sodium hydroxide**]** to further accelerate the expansion (as is commonly done). By then, the chemical composition of the concrete differed greatly from the concrete at Seabrook, and one could not use the cracking pattern or the expansion rates to be indicative of what would happen at Seabrook.

[redacted] **]**

In support of the above, suffice it to mention that:

- It is well established (Poyet, et al. 2007) that fine aggregates (sand) will yield a faster reaction (by virtue of their high volume to surface ratio which facilitates diffusion) than coarse ones. However, the coarse aggregates will ultimately yield larger expansion than the one caused by the sand. Hence, expansion will be under-estimated in the long run.
- Expansion is highly dependent on types of aggregates. Some are so called early-expansion, others are late-expansion. Overlooking the geological nature of the aggregate and sand will fatally compromise the outcome of any investigation.
- ASR field expansion as high as 3% have been reported in the literature. (Katayama, T. 2017). However, we have no idea of the potential ultimate expansion at Seabrook, because accelerated expansion tests were not performed.

All of the above will also have an impact in correlating crack widths, expansions, combined crack indexing (CCIs), and crack patterns with Seabrook. Last, but not least NextEra failed to use available tools that could have helped it to estimate the maximum expansion, such as accelerated expansion testing. In fact, the lack of subsequent accelerated expansion testing precludes NextEra from reaching any conclusions about the maximum likely degree of expansion.

In summary, it is not enough to have induced ASR expansion to claim that the concrete is representative.

C.2.2 Specimen dimensions, loads and boundary conditions were incorrect.

NextEra and its consultants made multiple errors with respect to the design of the tests (specimen dimension, loads, and boundary conditions).

C.2.2.1 Dimensions

A significant problem with the FSEL testing is the failure to ensure that the relative dimensions of the concrete beam that was tested were scaled to the prototype (*i.e.,* the Seabrook reactor). One of the basic principles of model testing is scaling. Before testing a model, one must first determine the largest dimension that can be accommodated in the laboratory (say x inches), and then determine the corresponding one in the prototype (in this case Seabrook) (most likely the thickness of the wall, say y inches). Then one would determine the scaling parameter alpha by taking the ratio of the two (y divided by x). This ratio should be respected in all other dimensional

quantities (especially reinforcement location and ratios) for a correctly-designed test. And the ratio will in turn govern the:

- Location of the reinforcement.
- The diameter of the reinforcement.

An example of a model next to a prototype is shown in Figure 2.

Figure 2 Example of a prototype and scaled down model from where a "test specimen" is extracted for laboratory testing.

But this was not done in the FSEL test program. The failure to scale the test models to the dimensions of the prototype prevents it from being representative in the significant respect of introducing the potential for an erroneous failure mechanism (a beam may fail by bending, or a combination of bending and shear; the degree of which depends on the relative dimensions and location of shear reinforcement). Under these conditions, the corresponding load will not be representative.

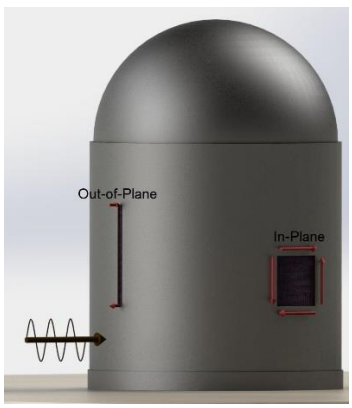
Hence, the test cannot be seen as a representative model of the prototype (Seabrook).

C.2.2.2 <u>Boundary Conditions</u>:

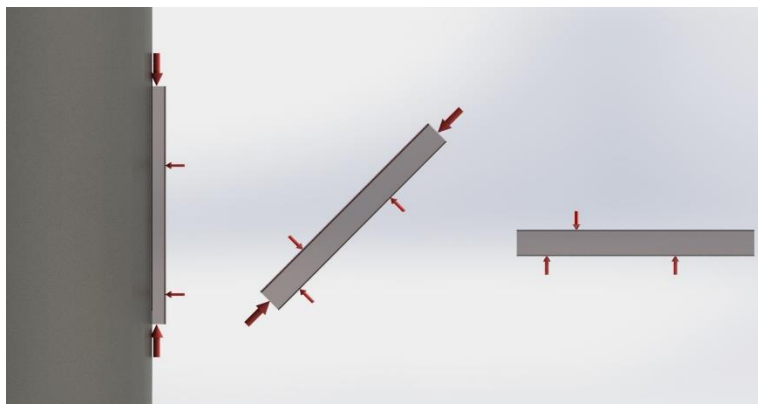
In a test, the model must be subjected to the same conditions (support, restraints and load) as the prototype (Seabrook). In this case, as shown by Figure 2:

- The FSEL tests modeled only the out of plane shear and not the in-plane. This can be visualized by holding a can in your hand and pushing in one direction. Along that direction the can must provide an out of plane shear resistance. But at 90 degrees, the can will be in in-plane shear mode. The later one was not tested. Out of plane results may not be directly applicable to in-plane. *See* Figure 3a.
- The beam (shown below in Figure 3b) was restrained from lateral expansion due to the adjacent concrete. This was accomplished by placing reinforcement as shown in Figure 7.
- The load has to mimic (to the extent possible) the lateral load caused by an earthquake. This is appropriately accomplished by the forces shown.

- However, the axial forces caused by the weight of the dome and the walls are not present in the beam. This pre-existing axial force has a strong influence on the shear response and will substantially negate the prestressing effect. Therefore:
    - The expansion in the vertical direction will be inhibited and redirected in the out of plane one where it has no impact on shear.
    - The magnitude of the ASR induced prestressing may be dwarfed by the pre-existing axial forces due to gravity load and hence cannot be relied upon.



a) Only out of plane shear (on the left) is tested. Impact of ASR on in-plane shear (on the right) is not assessed.



b) The beam is supposed to be representative of a segment of Seabrook. Hence, one can visualize the test as "peeling" away a beam out of it, and ensure that the beam will be subjected to the same loads as in Seabrook. In this case, one should also have an axial force (shown in the "rotating" beam), but by the time the beam is tested at FSEL the axial force is no longer present.

Figure 3 Boundary Conditions

As a result of these deficiencies, the FSEL test cannot be seen as a representative model of the prototype (Seabrook).

C.2.3 Failure to Address Load-Displacement, Cracking

C.2.3.1 Load Displacement

The objective of the FSEL test program was to determine the shear capacity of a beam affected by ASR. Hence, a beam was designed with sufficient reinforcement to resist bending, but not enough reinforcement to resist shear (this will induce shear failure).

If properly performed, cracking should have occurred in the zone without reinforcement (or the very little if present at Seabrook), and result in shear failure. This will result in a brittle response shown in Figure 5-a.

However, the FSEL test did NOT result in a crack. The reported load displacement curve is as shown below in Figure 4. **[**

EXHIBIT INT001

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[redacted]]

Figure 4 [[Confidential]] Reported Load-Deflection Curves

Figure 5.5, MPR-4273, Rev. 1 (Exh. INT021) (Proprietary)]

As shown, there is barely a "blip" (circled in Figure 4) and the curve proceeds. This is not indicative of a shear failure with minimum (or no reinforcement). Clearly, some shear reinforcement is present.

What is likely to have occurred is a crack in the zone of the beam with the shear reinforcement (as shown in Figure 5-b) which would cause a load displacement curve as the one reported. Thus, there is a very strong suspicion that shear did not occur where it was supposed to be, but in a reinforced region. Hence there was NO SHEAR FAILURE as intended. **[**

[redacted]]

Figure 5 "Shear" crack formation locations and corresponding load displacement response; Test probably was intended to have a crack in the top figure (green), but ended up occurring in the location shown below (red).

The authors should have included in such an important report pictures of shear cracks, similar to the one shown in Figure 6-a (based on a test with axial forces shown in Figure 6-b). Surely, they

were aware of the importance of such a picture as The FSEL (Deschenes, D. J., Bayrak, O., & Folliard, K. J. (2009) has published one (Fig. 3-10) in a report to the Texas Department of Transportation, Figure 7.

a) Example of a shear crack

b) Shear test with axial confinement (missing in FSEL tests)

Figure 6 Example of a shear crack

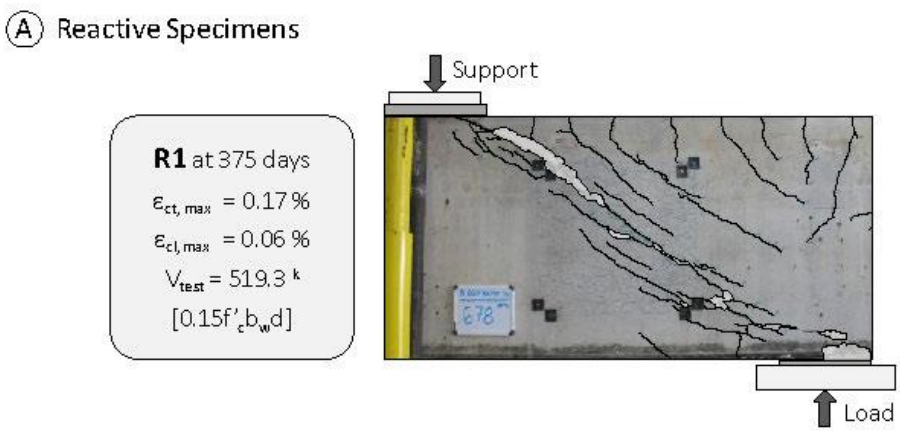

Figure 7 Example of shear crack picture reported by the FSEL Deschenes, D. J., Bayrak, O., & Folliard, K. J. (2009).

C.2.3.2 <u>Cracking</u>:

In its LAR, NextEra reported that by the time one of the test specimens was to be tested, it already had a longitudinal crack:

> *As ASR developed in the test specimens, a large crack was noted in the center of the surfaces of the beam that were between the reinforcement mats. This large crack is not representative of expansion behavior of structures at Seabrook Station, which have a*

> *network of members that are either cast together or integrally cast with special joint reinforcing details.*

MPF-4273, Rev. 1, p. 4-4 **(Exh. INT019).**

Such an unanticipated crack, shown in
]

Figure 8, should be of the utmost concern as it jeopardizes the representativeness of the ensuing test. **[**

[redacted]

]

Figure 8 Reported Large Crack from Surface between Reinforcement Mats

(Figure 4.2 in MPR-4273, Rev. 1 (Exh. INT021) (Proprietary))

There is a very simple explanation (regretfully, none was provided in the report) as to what most likely has happened. Figure 9 provides an illustration.

Expansion is restrained along the beam in the X and Y axis, but not in the Z axis. Hence ASR volumetric expansion was nearly entirely channeled into the Z direction and unsurprisingly cracked the beam. This is not unlike the delamination crack (between reinforcement mats) that occurred at Crystal River (though for entirely different cause), Figure 10.

Therefore, the specimen that was tested cannot be considered representative as it was already damaged, and ensuing results would unreliable. **[**

[redacted]

Figure 9 Possible explanation of crack splitting (B&W figure confidential, inspired by Fig. A-4 in SBK-PROP0001349)

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(Fig. A-4 in MPR-4273, Rev. 1 (Exh. INT021) (Proprietary))

Figure 10 Crystal River delamination (crack between the two reinforcing mats)

Hence, this test cannot be seen as a representative model of the prototype (Seabrook).

C.2.4 Erroneous assumptions regarding material properties and "design basis load"

C.2.4.1 Material Property

NextEra and its consultants made incorrect assumptions about ASR that led to serious errors in its analysis. They took a sophomoric approach that confused material strength with structural strength.

Concrete shear strength will decrease rather than increase because of ASR (as it is tightly related to the tensile strength widely known to decrease because of ASR).

Reinforced concrete, on the other hand, will not have a decrease in shear strength because of prestressing effect (restraint provided by the longitudinal reinforcement to crack formation).

The former is a universal material characteristic that can be used inside a finite element program's constitutive relation (a) to relate stress to strain; and b) to define a yield surface or failure load). The latter is specific to a structure, and cannot be used in a finite element analysis as it is specific to a structure. The only way NextEra could have extracted relevant information from their test to use in a finite element study would have been through a so-called system identification (Alves, S. W., & Hall, J. F. 2006). Simply put, one should distinguish material from structural tests (Lakes, 1993).

The error of confusing material strength with structural strength, as well as the other testing errors described above, became inputs to the finite element analysis relied on by NextEra for its safety assessment and monitoring program, and therefore have had a direct impact on the credibility of the finite element analysis. For these reasons alone, the finite element analysis.is not credible.

C.2.4.2 Capacity and Demand

NextEra also proposed to add ASR to its safety analysis as a "design basis load." SGH Report 160268-R-01, Rev. 1, Enclosure 4 to Letter SBK-L-16071 **(Exh. INT013).** The assumption that ASR can be considered a load is fundamentally wrong. In structural engineering, we must ensure that

capacity exceeds demand. Demand is the result of load (such as stresses). Capacity is the ability of the structure to resist a load by virtue to its strength (i.e., strength must be greater than stress). ASR affects capacity (reducing mechanical properties) and not demand. I do not know of any credible example where ASR is defined as a load. This is a serious error in NextEra's approach to ASR.

C.2.4.3 ASR Load Factors

ASR load factors were used in the analysis and are determined in the LAR **(Exh. INT010).** Their determination hinged heavily on crack indexing (CI) (surface) measurements that I consider misleading. Hence the load factors are not reliable.

C.3 Monitoring

As a structural engineer, I find it very difficult to sharply distinguish between AAR monitoring and structural monitoring. One cannot and should not separate material from structural effects, as the two are intertwined. To ignore this fact and decouple them is a grave mistake, as demonstrated in Figure 11.

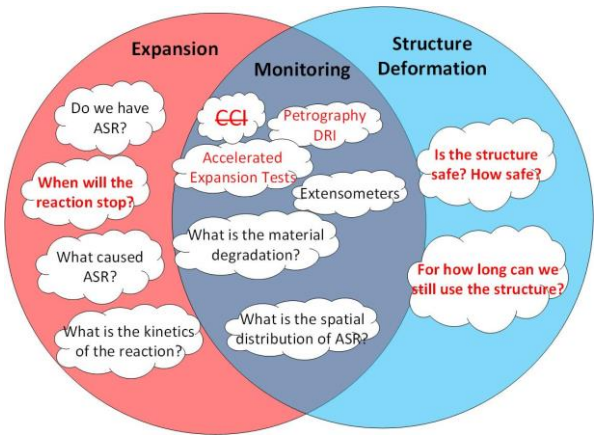


Figure 11 Interconnection between AAR and Structural Monitoring

In addition, both the ASR monitoring program and the Structural Deformation Monitoring Program are based in part on the faulty FSEL test results. As a result, both programs prescribe monitoring measures that are insufficient to address the actual conditions at Seabrook. And as shown in Figure 11, these defects are interdependent.

Nevertheless, this will be subdivided into two parts: the physical monitoring and the structural analysis component.

C.3.1 Can you comment on the expansion monitoring?

C.3.1.1 Volumetric Expansions

At page 55, the NRC's Safety Evaluation (**Exh. INT024)** makes several statements that are not supportable:

1. *Volumetric expansion is also calculated (sum of measured expansion in two in-plane directions and the through-thickness direction) and compared to a limit based on the MPR/FSEL LSTP results. The NRC staff finds this inspection approach, and the associated inspection methods, acceptable.*

This is where there is a direct connection between FSEL measurements and Seabrook. Again, FSEL measurements cannot be applied at Seabrook for reasons explained below.

2. *Ultimately, volumetric expansion is monitored and compared to conservative limits determined during the MPR/FSEL LSTP*.

There is no basis for what constitutes "conservative limits measured in the MPR/FSEL LSTP program." Again, conditions are quite different.

3. *The progression of inspection methods (visual to CCI (or Cl/CCI supplemented by pin-to-pin expansion measurements) to through-wall expansion) ensures that ASR degradation is identified as soon as reasonably possible and that the degradation is monitored as it progresses to ensure that impacted structures remain functional*.

Again, as explained above, ASR is NOT monitored as it progresses.

C.3.1.2 Role of CCI

As described in the May 2018 revised LAR (Letter SBK-L-18072, Encl. 1, p. 3) **(Exh. INT016)** NextEra's measurements methodology can be summarized as follows:

1. Rely on visual inspection, look for gel exude, moisture, crack "pattern" (tier 1).
2. Measure CCI (as "*CCI correlates well with strain in the in-plane directions and the ability to visually detect cracking in exposed surfaces making it an effective initial detection parameter*"). If less than 0.1% on the surface, monitor qualitatively and quantitatively (tier 2).
3. If CCI is greater than 0.1% (tier 3):
    a. Structural Evaluation
    b. Install through wall expansion monitoring with extensometer.
    c. Determine volumetric expansion (adding extensometer and CCI values)

Enclosure 2 to Letter SBK-L-18072, page 7 **(Exh. INT017)** states that tests were completed at various levels of ASR cracking to assess the impact on selected limit states. The extent of ASR cracking in the test specimens was quantified by measuring the expansion in the in-plane and

through-thickness dimensions. The in-plane dimension refers to measurements taken in a plane parallel to the underlying reinforcement bars.

Hence, field measurements hinge on FSEL readings for calibration/validation.

Furthermore, CCI has been justified by reference to a Federal highway administration report. However, inspection of Figure 12 of this FHWA report by Fournier, B., Berube, M. A., Folliard, K. J., & Thomas, M. (2010) in Sect. 2.2. indicates that CCI can only be used in conjunction with petrography for Level 2:

> *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}. Clearly safety investigation of the impact of ASR during ASR would necessitate a Level 3 investigation.*

Based on the above reputable report, In the context of Seabrook, CCI should be used in conjunction with continuous/additional petrographic studies.

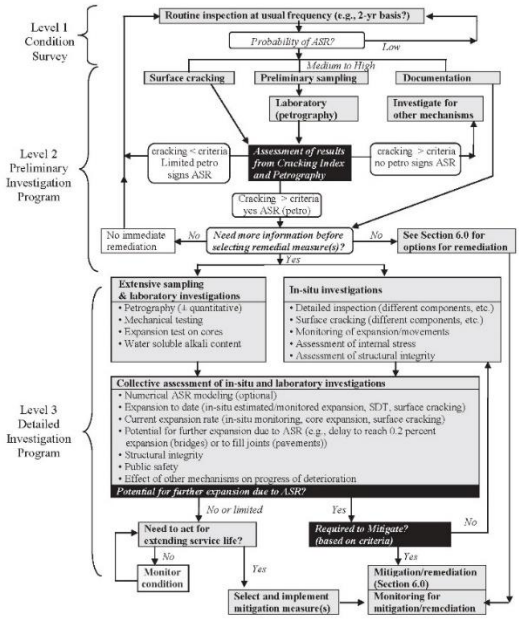


Figure 12 FHWA report by Fournier, B., Berube, M. A., Folliard, K. J., & Thomas, M. (2010).

Sect. 4.2.6 of Enclosure 3 to Letter SBK-L-16071 **(Exh. INT010)** indicated that "*ASR proceeds more rapidly in hot and moist conditions. Test specimens were stored in an Environmental Conditioning Facility (ECF) with alternating wet and dry cycles to promote ASR development.*"

In this context, it should be kept in mind that the relative humidity (RH) on the surface of a concrete dam or wall a few inches below the surface is well below the 80% threshold for AAR to occur (Stark, et al., 1987). This is implicitly recognized in the report (Sect. 4.2.6 of the LAR (**Exh.**

INT010)) as the test specimens were stored in an Environmental Conditioning Facility (ECF) because "*ASR proceeds more rapidly in hot and moist conditions*".

Whereas Starks based his measurements in dams, they are very pertinent to Seabrook:

1. In New Hampshire, temperature is much lower on the surface of the wall, and there is a thermal gradient with the much warmer concrete inside.
2. The surface of the walls has dried due to shrinkage long time ago, and the relative humidity is much reduced. However, the inside of the concrete would maintain a high one (water to cement ratio is nearly always higher than the one needed for concrete hydration).

Both effects reduce the reliability of CCI.

Hence, should cracking be noticeable (through the CCI), that would imply that the internal swelling (where the RH is higher than 80%) was so great that it affected the surface cracking. By the time extensometers are installed, it may already be too late, Figure 13.

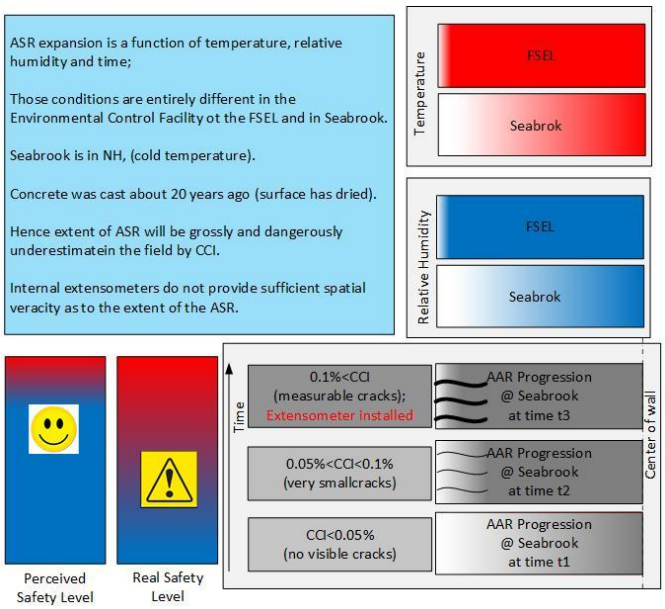


Figure 13 Progression of ASR and likelihood of being captured by CCI

Therefore, I conclude that as a monitoring measure, CCI must be ruled out completely.

C.3.2 <u>What other physical in-situ measurements should be employed</u>?

Embedded extensometer (though installed possibly too late) for the out of plane strain should be placed at least at mid-distance between the intrados and extrados (there is no indication as to which depth they were inserted).

Internal relative humidity should be measured with wood stick method, or Capacitance probes, or a microwave technique (Ground Penetrating Radar, Time Domain Reflectometry, or Open-ended coaxial probe). This is important, as no expansion would occur when the RH is below 80%.

Because of the proximity of the sea, concrete should be tested for its (free) chloride concentration and make sure that it is below critical limits before steel depasivate (i.e. corrode).

The basemat should be inspected (as close to the center as possible for cracks and misalignment with respect to the walls). Because it is (or has been) below the water table for an extended period, the high relative humidity may have enhanced the likelihood of ASR occurrence,

Figure 16-a.

C.3.3 Can you comment on Expansion Monitoring as it relates to Structural Deformation Monitoring?

This is a most confusing distinction made by NextEra. Whereas expansion monitoring clearly refers to ASR expansion measured by CCI and other methods, what is meant by "Deformation"? Strictly speaking, deformation monitoring implies field measurement of displacements, and nothing else. Clear and simple.

To the extent I was able to make some sense of this part, my understanding is that the Structural Deformation Monitoring relates primarily to the monitoring intervals as summarized in Table 6 of their report and shown in Figure 14.

Stage	Deformation Evaluation Stage	Monitoring Interval
1	Screening Evaluation	3 years
2	Analytical Evaluation	18 months
3	Detailed Evaluation	6 months

Figure 14 Structure deformation monitoring requirements

(Letter SBK-L-18072, Enclosure 2, page 34 **(Exh. INT017)**)

C.3.4 Can you comment on the role of the finite element analysis as it relates to expansion monitoring?

Table 5 of Letter SBK-L-16071 **(Exh. INT010),** as adapted here in Figure 15, illustrates NextEra monitoring approach. The grayed portion (by me) relates to the frequency and pertains to Structural Displacement Monitoring (as discussed above), and hence will be ignored by me.

However, I would call your attention to the highlighted part "Structural evaluation" in Tier 3.

Tier	Recommendation from Inspection	In-Plane Expansion	Inspection Frequency
1	Routine inspection in accordance with SMP	NA*	As prescribed in the SMP
2	Qualitative monitoring	Areas with pattern cracking that cannot be accurately measured	30 months
	Quantitative monitoring and trending	0.05%	
3	Structural evaluation and implement enhanced ASR monitoring	0.1%	6 months

* No indications of pattern cracking or water ingress.

Figure 15 ASR Expansion Acceptance Criteria and Condition Monitoring Frequencies

adapted from Letter SBK-16071, Table 5 ***(Exh. INT010)****)*

Under Sect. 3.3.2 of Letter SBK-L-16071 **(Exh. INT010),** entitled *Evaluation of Self-Straining Loads and Deformations for Seismic Category I Structures other than Containment* (more specifically Stage Three: Detailed Evaluation), the LAR states:

> *In the Detailed Evaluation, S_a demands and the loads from creep, shrinkage and swelling are recomputed using the Stage Two FEM. Structural demands due to design loads are recomputed by applying design demands (i.e. wind, seismic, hydrostatic pressure, etc.) to the FEM. A detailed structural evaluation is performed for all load combinations listed in UFSAR Table 3.8-16. The structure is evaluated using strength acceptance criteria in ACI 318-71 for reinforced concrete…*

Furthermore, in Sect. 3.5.2 Structure Deformation (and not structure deformation Monitoring), the LAR states:

> *The SMP also includes the requirements for monitoring Seabrook structures with measureable deformation. Structures with ASR are initially screened for deformation using the process described in Section 3.3. The process will classify affected structures into one of three categories: (1) structures with minimal amounts of deformation that do not affect the structural capacity as determined in the original design analysis; (2) structures with elevated levels of deformation that are shown to be acceptable using FEA and still meet the original design basis requirements when ASR effects are included; and (3) structures*

with significant deformation that are analyzed and shown to meet the requirements of the code of record using the methods described herein.

Hence, the structural assessment is an integral part of ASR expansion monitoring, and the finite element analysis in turn is an integral part of the structural assessment.

Having established the connection between static and dynamic finite element analysis to expansion monitoring, I would like to break my comments into two parts:

- Static and dynamic finite element analysis
- Probabilistic vs Deterministic paradigms.

The serious concerns raised by the finite element procedure have been extensively addressed in my expert report **(Exh. INT007)**, and only major findings will be mentioned here.

C.3.4.1 *Why are you so concerned about the structural evaluation as embodied by the finite element analysis with ANSYS?*

ANSYS is an excellent tool, but it has its limitations (explicitly modelling ASR by accounting for all its idiosyncrasies detailed above), and as any other program it can be misused.

Based on the methodology described in Simpson Gumpertz & Heger, Inc., "Evaluation and Design Confirmation of As-Deformed CEB, 150252-CA-02," Revision 0, July 2016 (Seabrook FP#100985) **(Exh. INT015)**, I have the following concerns:

1. **pg. i** *Seismic loads are applied using a static equivalent method utilizing the design-basis maximum acceleration profiles, which were computed during original design from response spectra analysis. Amplify ASR loads by a threshold factor to account for potential future ASR expansion. Evaluate capacity based on ACI 318-71 criteria with combined demands from all design loads, including the self-straining loads associated with the as deformed condition*.

The concern of a linear elastic analysis for such a critical safety assessment have been previously mentioned, Figure 1.

2. **pg. 14** *Alkali-silica reaction (ASR) demands are selected based on extensive field measurements of strain on the CEB [9] and are increased by a load factor to account for uncertainty in the demands and a threshold factor to account for limited future ASR expansion***.** And t*he strains due to ASR expansion simulated by the finite element model (FEM) reasonably approximate crack index measurements*

This was again addressed above. NextEra is confusing Capacity with Demand. ASR reduces capacity and whereas it is a stressor, it is not a load to be amplified in the Demand.

3. **pg. 16** *All ASR loads are amplified by a threshold factor of 1.2 in addition to the load factors for ASR. The threshold factor accounts for additional ASR loads that may occur in the future*.

Whereas the premises of this statement are erroneous (previous point), on what basis is it assumed that future expansion will increase by 20%? How is "future" defined? This could have been partially validated by accelerated expansion tests,

Figure 18, had they been performed.

4. **pg. *18*** *The elastic modulus of concrete is not reduced due to ASR damage*

This is erroneous. There is no doubt that the elastic modulus E is affected by ASR, and this will in turn result in larger displacements, and in turn increased likelihood of cracking which is precisely what one wants to avoid, Figure 1.

5. **pg. 22, JA02** *The magnitude of ASR expansion (and the associated tensile and compressive forces) used in this evaluation and design confirmation is based on field measurements*.

Again, this is where monitoring and structural assessment intersect. The representativeness of tests has been challenged in Sect. C.2.1

6. **pg. 22, JA03** *Unreduced design material stiffness properties can adequately represent ASR impacted reinforced concrete sections of the CEB structure... Therefore, an unreduced elastic modulus based on the design concrete compression strength f'c is used in the Standard and Standard-Plus Analysis Cases in this calculation*.

Again, the elastic modulus should have been reduced, and this in turn will reduce the stiffness of the NCVS. Indeed ACI-318 (section 19.2.2.1) has an approximate equation for the elastic modulus in terms of the compressive strength. However, this cannot be valid for a deteriorated concrete as it is outside the assumptions of the ACI equation.

7. **pg. 22, JA04** *However, the same aggregate source was used for the concrete fill as for the CEB concrete*.

This is incorrect, as is stated separately in MPR-4273, Rev. 1, page 3-2 (**Exh. INT021) (Proprietary):**

[redacted] **]**

Furthermore, the source of the sand (though it is mentioned that highly reactive one was selected) is unknown. In other words, one cannot reasonably assume that the tests had the same concrete as Seabrook.

8. **pg. 25, JA11** *ASR expansion impacts the total demand on reinforced concrete elements, but does not reduce the resistance (capacity) of reinforced concrete elements so long as the strain does not exceed the limits defined in Ref. 16*.

This is incorrect. The concrete material is degraded by ASR (by virtue of its correlation to the tensile strength). Again, there is confusion between structural testing (indeed ASR may increase the strength) and material testing (where it does not) needed for a finite element analysis. Furthermore, Strains in the linear elastic analysis (performed) are grossly underestimated.

9. **pg. 31** *The CEB walls and dome concrete consist of four-node shell elements ... modeled using centerline geometry.*

The shell elements used in the finite element study could be a reasonable approximation under different circumstances. However, it cannot capture the through thickness expansion which is lower on the surfaces and higher in the center (different RH), Given the nature of the problem, one would have thought that solid 3D elements would be used for a more accurate modeling.

10. **pg. *32*** *The base of the CEB foundation is restrained vertically... Since ASR expansion of the wall is largest below-grade*.

ASR was modeled in that portion of the concrete below grade as it is that portion most likely to have been in contact with water. Hence, one would have to assume that the base mat will also suffer from ASR. This will result in a “bubble'” expansion with corresponding lift-off in the middle, whereas on the periphery the walls provide sufficient restraint,

Figure 16-a.

11. **pg. 37** *Varying magnitudes of ASR expansion are applied to the CEB finite element model based on field measurements of Cl.*

Again, wrong. As previously illustrated, Figure 13 the crack index is a most unreliable indicator of ASR, and no serious researcher would rely on it.

12. **pg. 40** *ASR expansion is simulated by applying a thermal expansion to the elements representing the CEB concrete. ...The steel membrane elements are only included in the model when applying ASR expansion of the CEB wall and concrete swelling.*

These are some of the most troubling assumptions of the entire analysis. Whereas many, many years ago, this was the simplest way of modeling ASR, by now it is no longer in use. A simplistic thermal expansion will fail to capture the anisotropic nature of the expansion (in this case the preponderance for the expansion to be out of plane and not in plane). Difficult to understand why one would have to include steel only for the ASR study and not for the other load cases. Either steel is present or not, entire analysis has to be performed consistently. Furthermore, the kinetics of the reaction is not captured, and no future predictions could be made.

13. **pg. 41** *Research referenced by this assessment indicates that unreinforced concrete (if in conditions similar to the CEB) can be expected to swell approximately 0.02% and reinforced concrete can be expected to swell by approximately 0.01 %.*

This is completely arbitrary. There is no scientific basis for such range of values. Expansions vary depending on the source of aggregates, the alkalinity of the cement, the relative humidity, the temperature, and the state of stress. This is a very dangerous and simplistic assumption with no basis.

14. **pg. 44** *Response spectra analysis was performed using a simplified "stick" model. For lateral analyses, the model was fully fixed below El. 0 ft. For vertical analyses, the model was fixed at the base at El. (-)30 ft.*

Again, the stick model is a model of the past when computers did not have sufficient capability to handle the time history analysis of a 3D model, Figure 16-b. The stick model is overly simplistic, as discussed in the quote below:

> *The dynamic analysis of containment structures for earthquake loads have progressed from a few two-dimensional lumped three or four mass stick models employing response spectrum modal analysis (in the late 1960s) to complex three-dimensional hundreds to thousands of degrees of freedom finite element models (in the 1970s and 1980s). The dynamic modeling of containment has generally included Soil-Structure Interaction (SSI) effects.* (Hansraj Ashar et al. 2001).

Furthermore, stick model cannot capture the seismic contact between the wall and the adjacent soil unless joint elements are inserted.

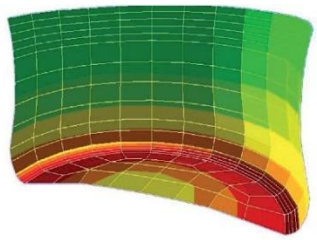
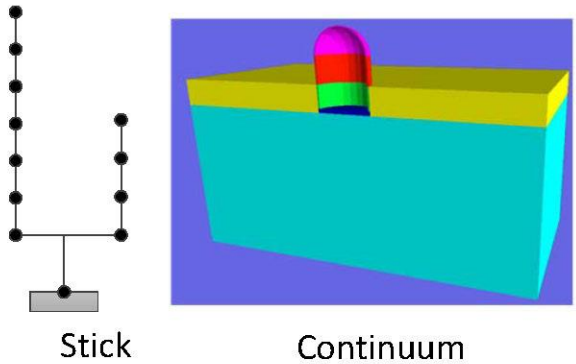
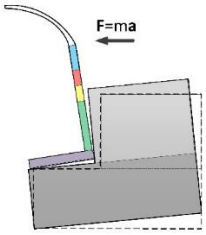
		
a) Anticipated deformation of the base mat (note upward deflection)	b) Stick model used by NextEra compared with the current State of Practice continuum model	c) Potential uplift modeled by joint elements

Figure 16 Some ignored considerations in the analysis that may severely impact results

15. **pg. 52** *Structural capacities are evaluated for all analysis ... using the element-by-element approach ... as well as the section cut approach... Evaluation criteria for strength of reinforced concrete components are taken from ACI 318-71 [11]. The threshold factor, which amplifies ASR demands to account for future ASR expansion*

The report takes a complex structure (subjected to ASR and in some cases to seismic excitation) and reduces its assessment to mere column subjected to combined axial forces and moments through the interaction diagram established for structural component design. This approach reduces the NCVS to a series of parallel column with no interaction among them. One should have examined indeed element by element and assess strength through established failure criteria applicable at that location (Mohr-Coulomb for concrete, yield for steel). Likewise, serviceability (cracking under service loads) can only be quantified through a nonlinear analysis capable of capturing cracks.

16. **pg. 52** *Evaluating a structure on an element-by-element basis is considered a conservative approach because it does not allow for concentrations of high demands to be distributed locally within the structure. Factored demand exceeding capacity in the element-by-element evaluation does not necessarily indicate a structural deficiency.*

There is no basis for such a statement. A nonlinear analysis should have been performed.

17. **pg. F-3** *Cracked section properties do not affect the global seismic response of the CEB. This assumption is justified because the global response of the CEB to seismic motion*

primarily causes in-plane shear and overturning stresses; both are resisted by the membrane stiffnesses of the CEB wall that are not impacted by cracking.

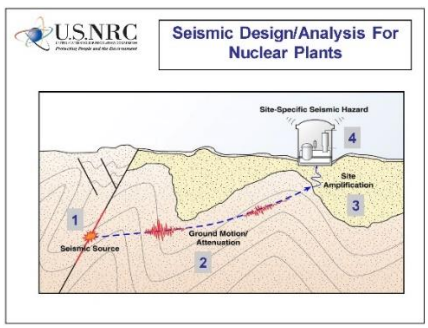
Tests were performed for out of plane shear, and they were indeed criticized because results were also applied to zone in-plane shear. This is best illustrated by Figure 3-a.

This also implies that cracks will only occur along the direction of the seismic excitation. This is wrong, and shows a poor understanding of structural response of a cylinder subjected to shear force.

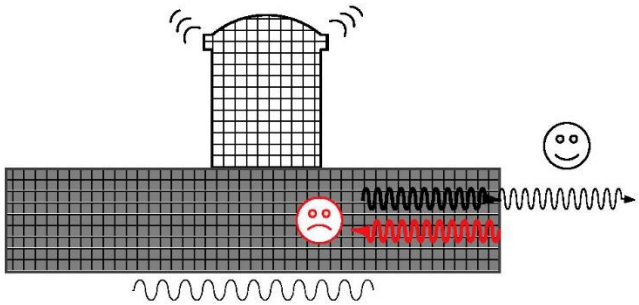
18. **pg. K-5** *Compute axial strain in concrete due to as-deformed condition*.

Because of the linear elastic analysis, the strains are grossly underestimated, best illustrated by Figure 1.

19. Impact of soil through proper deconvolution or soil structure interaction is not accounted for as required by ASCE 4-16 (2016),
20. Figure 17.



a) Seismic Attenuation/Amplification J. Ake, J. Pires, and C. Munson (2015)



b) Need for soil-structure interaction in seismic analysis

Figure 17 Not modeled impact of soil in the seismic analysis

C.3.4.1.1 <u>Why should a Probabilistic Based Analysis be performed for the structural evaluation?</u>

Given the high risks associated with an accident at Seabrook, given the uncertainties associated with Capacity (concrete properties primarily) and Demand (seismic excitation primarily), a structural assessment model that accounts for these uncertainties must be used.

This assertion is supported by the following:

- Nature is random, and so are concrete properties and loads. Already SG&H Report 160268-R-01 (Enclosure 4 to SBK-L-16071) **(Exh. INT013)** explicitly recognizes that

randomness and introduces the concept of Reliability Index in Sect. 1.4.4. This is a target value (typically around 3.5) and the ACI code determines the load factors to the Demand (and a reduction factor to the Capacity) such that this value is met.

The reliability index assumes a normal distribution for Demand and another to Capacity, Failure is defined when the former exceeds the second.

Hence, the probabilistic approach is already enshrined in the analyses reported by NextEra.

- Furthermore, as stated by the American Society of Civil Engineers:

  Regulatory government agencies are frequently faced with decisions related to the seismic design of operating nuclear facilities... As new information becomes available, the design basis may be challenged. ... Because of its pervasive nature, an earthquake will "seek out" facility vulnerabilities...At issue is whether the changes can be accommodated within the inherent capacity of the original design or whether facility modifications are required.... current design practice does not provide a picture of the actual margin to failure, nor does it provide enough information to make realistic estimates of seismic risk... The seismic probabilistic risk assessment (SPRA) is an integrated process that includes consideration of the uncertainty and randomness of the seismic hazard, structural response, and material capacity parameters to give a probabilistic assessment of risk.

Seismic Analysis of Safety-Related Nuclear Structures (American Society of Civil Engineers 4-16, 2016)

Clearly, and unequivocally, the presence of ASR is a "new information" and it challenges the design basis. But what is probabilistic risk assessment? Beckjord, Cunningham, and Murphy (1993) defined it as:

> *Probabilistic risk (or safety) assessment (PRA) consists of an analysis of the operations of a particular nuclear power plant (NPP), which focuses on the failures or faults that can occur to components, systems or structures, and that can lead to damage and ultimately to the release of radioactive material, especially the fission products and actinides within the reactor fuel.*

Probabilistic methodology is a procedure pioneered by the NRC starting with the landmark NRC report (Wash-1400) cast the foundations for probabilistic risk assessments. Then, following the 1979 accident at Three Mile Island, it was recommended that PRA be used to complement the traditional deterministic methods of analyzing nuclear power plant safety and that probabilistic safety goals be developed for nuclear plants Kemeny (1979), NRC (1987). Finally, after Fukushima, plants were required to reevaluate the potential impact of external events on their structures (EPRI 2013) as in some cases the seismic stressor may have been underestimated (Hardy et al. (2015).

In recent years, there has been a gradual shift toward probabilistic risk evaluations. Kennedy and Ravindra (1984) were the first to introduce the concept of seismic fragility for NCVS. Fragility curves are conditional failure frequency curves plotted against peak ground acceleration (PGA). This general framework accounted for both aleatory and epistemic uncertainties.

Seabrook should be investigated through this angle and not through a 1971 design code.

C.4 How do the problematic FSEL test results affect NextEra's safety assessment for ASR at Seabrook?

The FSEL tests compromise the reliability of NextEra's safety assessment in multiple ways:

1. Reliability of crack observations and expansion measurements. Because of the very different concrete mix, one cannot rely on FSEL data to quantitatively and properly interpret field measurements at Seabrook.
2. The CCI method developed in the laboratory under the special environmental condition of high humidity is not applicable in Seabrook. In Seabrook, the concrete has dried on the surface and there will be very little cracking as a result of ASR.
3. NextEra has prematurely ruled out the applicability of petrographic DRI. based on results obtained. When properly done, DRI remains a widely recognized diagnostics assessment tool and should be monitored with time. I would caution that this is a delicate test that should only be performed by a very qualified petrographer, and should be performed repeatedly by the same one.
4. As explained in the previous section, because the FSEL tests were so defective, their results cannot be used by any finite element code to gauge the safety of the structure (as they have been used by SG&H).

C.5 Are there other tests that could have been performed to characterize the ASR?

There are three critical questions confronting an engineer overseeing the safety of a structure affected by AAR:

1. How much time would elapse before the reaction stops.
2. What would be the maximum expansion at the time the reaction stops.
3. How would that affect the safety of structures under consideration.

Answers to the first two question can be estimated through a combination of good petrographic analysis (to estimate past expansion), and an accelerated expansion test. This is a very commonly performed test (and recommended by the FHWA). Analogous tests were performed on concrete blocks at the University of Texas Austin) by Dr. Kevin Folliard, a world-renowned expert in ASR at the University of Texas, Austin (Fournier, et al. 2006, and thus should have been known (and consulted) by Prof. Bayrak. In fact, Fournier, et al. 2006 is one of the most cited ASR references according to Google Scholar.

This test is described below in C.6, and Figure 18.

C.6 Based on your experience, what steps would be necessary to address ASR at Seabrook?

ASR is very complex phenomena, one that may result in counter-intuitive observations. Hence, extraordinary problems (by virtue of their complexity and safety impact) demand extraordinarily rigorous methods of investigation.

Hence, the first approach should be to get familiar with the state of the art, and not to oversimplify things.

For an appropriate assessment of safety, it is also essential to distinguish design of new structures from analysis of existing ones. Design is simple, linear and elastic. Safety assessment analysis is nonlinear and far more complex. It is important not to perform a safety assessment analysis of an existing structure in 2019 as if you were designing a new one and use a 1971 design code.

It is also essential to take steps to get good data. This is also a complex process requiring proper testing. First and foremost among them is the use of accelerated expansion tests. This is a well-established test procedure not only for mortar bars but also for concrete. Because AAR is a thermodynamically driven reaction, it can be accelerated by storing cores at temperatures ranging from 38 to 60 deg C. Small "disks" are glued on the cores, the cores are then placed in a container, and the container in a so called reactor which is heated to the right temperature,

Figure 18-a).The cores are periodically extracted, and the elongation is measured with a so-called DEMC instrument between the disks.

An outcome of this test is a plot similar to the one in

Figure 18-b. The test will stop when the expansion has "plateaued" and will provide the maximum expansion likely to occur, and as importantly the kinetics of the reaction, that is when the maximum expansion is likely to occur (this will take into account the temperatures of the reactor and of Seabrook). This test should be accompanied by a detailed petrographic study which will give an indication of past expansion and give the geologic nature of constituents. This last point is important to determine if one is dealing with early or late expansion types of aggregates

EXHIBIT INT001

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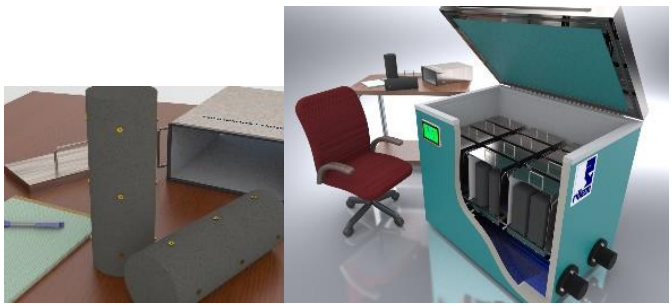
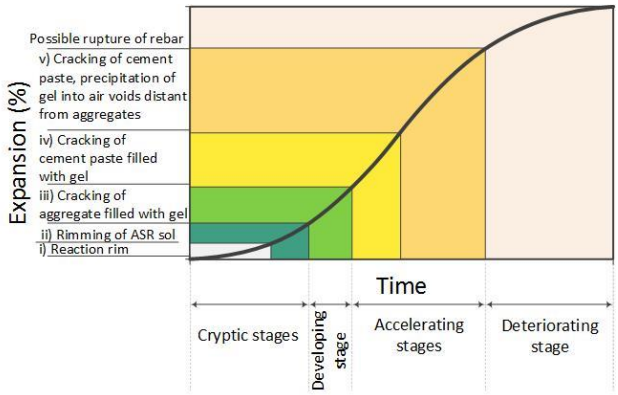
	a) Cores next to a container, and containers inside a reactor.
	b) Sigmoid curve representative of AAR expansion that can be obtained from accelerated expansion tests.

Figure 18 Elements of accelerated expansion test

- Site
  - Properly placed extensometers (deep inside the wall).
  - Record temperature, relative humidity, and free chloride (because part of the structure may have been submerged under sea water)
  - CCI measured on the surface are not indicative of ASR expansion. At best one would be measuring the shrinkage cracks (given that the surface is dry), and by the time AAR expansion is large enough to further open the surface crack it may be too late to correctly capture the extent of the reaction.
- Data should be input to a finite element simulation using proper modeling of ASR. At a minimum, data should be obtained and modeled to account for:
  - Induced expansion anisotropy.
  - Dependence on relative humidity and temperature (and those are not constant across the wall thickness).
  - Mechanical degradation of the concrete (elastic modulus, tensile, compressive and shear strength, fracture energy).
  - Stress redistribution.
  - Reliance on laboratory tests described above.

- Validation/calibration with site measurements.
- Include modeling of uncertainties.
- Seismic analysis brings in its own complexities.

C.7 Has this approach been followed in evaluating other major structures suffering from ASR?

Yes, I have observed or participated in multiple projects where an appropriate degree of rigor and sophistication was used. Personally, I have assisted TEPCO to assess the safety of a massive reinforced concrete structure subjected to ASR and then hit by a major seismic excitation. Likewise, I have assisted the Swiss Dam Safety Office to assess the safety of Isola dam suffering of ASR.

I am also currently engaged in a three-year evaluation of ASR at the Seminoe dam for the U.S. Dept. of the Interior Bureau of Reclamation. The problem of ASR originally was analyzed by a major engineering firm using a simplistic approach. Results were deemed unconvincing. Therefore, the Bureau of Reclamation has decided to take a pause to study the State of the Art in ASR for three years, and then revisit the dam with modern analytical methods. This illustrates the importance of seeking the expertise of leading researchers in the absence of established standards for evaluating the hazards posed by ASR.

Other examples include Gentilly 2 and Ikatu nuclear containment structures in Canada and Japan. They were scientifically investigated with the tools of the days. (J. Tcherner and T. Aziz, 2009), (Murazumi et al., 2005a). Finally, there are numerous examples of bridges and dams suffering from ASR where the technical expertise of leading researchers was sought before action/decisions were taken (mostly in Europe and Japan, to a lesser extent in the U.S.). For instance, in Japan, rebars of a bridge suffering from ASR snapped and broke. This led to numerous scientific papers to properly understand what happened. In Switzerland, the famous Viaduct of Chillon above Montreux exhibited signs of ASR cracks. This has prompted the Swiss authorities to perform numerous petrographic studies, and most importantly accelerated expansion tests (which were not reported for Seabrook).

In addition, Mactacuaq dam is severely affected by AAR, and presently New Brunswick Hydro is trying to determine whether to replace it (a ~$4B cost) or keep it under observation. For now it is under observation using far more sophisticated tools than the one used by NextEra (in terms of monitoring and finite element simulation).

C.8 Do you have other recommendations with respect to elements that should be included in an ASR monitoring program?

ASR results in reduced Young's modulus, and is highly dependent on relative humidity. Those parameters are not monitored by NextEra, but they should be monitored.

EXHIBIT INT001

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The following table describes various monitoring methods. Extracted from NDE chapter (written by Alexis Courtois (EdF, DTG), Dr. Eric R. Giannini (R J Lee Group), Alexandre Boule (EdF, DI), Jean-Marie Hénault (EdF R&D) , Prof. Laurence Jacob (Georgia Tech), Benoit Masson (EdF DTG), Prof. Patrice Rivard (Laval University), Jerome Sausse (EdF, DTG) and Denis Vautrin (EdF R&D) in a soon to be published book "Diagnosis and Prognosis of ASR Affected Structures" edited by Saouma.

Summary table of techniques for monitoring ASR-affected structures

Method	POC or PAT*	Accuracy for ASR Diagnosis
Young modulus, local stiffness		
Ultrasonic Pulse Echo (UPE)	PAT	B**
Ultrasonic Pulse Velocity (UPV)	POC	C**
Impact-Echo	POC	C
Acoustic Emission (AE)	PAT	C
Promising techniques with high resolution and high sensitivity • High sensitivity • Nonlinear acoustic • Diffusion Surface waves	PAT	A**
Concrete moisture/humidity/water content		
RH/Capacitance Probe	POC	C
Wood Stick	POC	B
Microwave Technique: GPR	PAT	B
Microwave Technique: TDR	POC	B
Microwave Technique: Open-Ended Coaxial Probe	PAT	B

* POC: Proof-of-Concept for structural monitoring of ASR-relevant parameters on real structures.
PAT: Potentially Applicable Technique for monitoring ASR-relevant parameters, but not performed with success yet at the structural level in the field.
**A: promising, high sensitivity, B: needs calibration on the tested concrete, C: specific care to avoid possible leaks

I would suggest that a combination of Petrographic and accelerated expansion tests be independently performed by three of the world leading experts:

1. Dr. Andreas Leemann, EMPA, Switzerland.
2. Dr. Tetsuya Katayama.
3. Dr. David Rothstein

I would also propose that a probabilistic based nonlinear finite element analysis be performed. The finite element code should be, to the extent possible, ``validated'' by the benchmark problems set forth by the RILEM TC-59 committee for both static and dynamic loads. The finite element code could be further validated by simulating the response of the beams tested by the FSEL before it analyses Seabrook.

C.9 How important is peer review in the steps you advocate?

Independent peer review is a cornerstone of engineering practice. It is of paramount importance that the reviewers be sufficiently detached from the project organization, *i.e.,* they do not ultimately report to the same hierarchy. And peer reviewers should be familiar with the literature. Finally, they should have a degree of scientific expertise and rigor that is sufficient to enable them to credibly comment.

In this case, NRC used the term "peer review" but only to describe a review by other employees of the agency who did not have immediate responsibility for the analysis of ASR at Seabrook. Such a review does not have the independence required for a peer review, nor the necessary expertise in the specific area of ASR.

Last, but not least, it is evident that the P.I. of the FSEL tests may have much experience in large scale testing, but at the time the tests were conducted, he had (according to Google Scholar) no more than one or two peer-reviewed scientific publications related to ASR (and that was for testing). In other words, there was no in-house expertise on ASR, though an internationally renowned expert (Prof. Foilliard) is at the same institution. In other words, the complexity of ASR was not recognized by the FSEL test team.

C.10 In your opinion, is the license extension for Seabrook justified?

No, I do not believe the work done by NextEra to evaluate ASR and establish a monitoring program is sufficiently reliable or sophisticated to support a finding of regulatory compliance. More appropriate assessment studies and monitoring programs should be put in place immediately.

C.11 Finally, can you please tabulate all your concerns for ease of reference?

Yes. They are:

1. Concrete mix design very different from the one of Seabrook, cannot be deemed as representative
2. Lack of Accelerated Expansion Tests.
3. Very limited usage of Petrography (DRI in particular).
4. Specimen design (dimensions and internal reinforcement) is not a model of the prototype (Seabrook).
5. Splitting test occurring before shear tests negates validity of test.
6. Erroneous boundary conditions.
7. No evidence of a shear failure.
8. Lack of images to identify/confirm occurrence of shear failure.
9. Surface measurements of CI/CCI misleading.
10. Should measure free chloride content as well as temperature and relative humidity inside the walls.
11. Design and safety assessment should use different methodologies of investigation.
12. ASR is not part of the Demand (load) but part of the (ability of the concrete to resist stresses);
13. Load factors used in the analyses are unreliable because they are based on CI measurements, which are unreliable themselves.
14. ASR cannot be assumed to be uniformly spread across the thickness of the wall.
15. ASR's expansion cannot be assumed isotropic and independent of the state of the stress.
16. ASR is not temperature-independent.
17. ASR and its impact cannot be analyzed linear elastically.
18. Stick model for dynamic analysis is obsolete and cannot capture response.
19. Nature is nonlinear and random. Those must be accounted for.
20. Absence of independent peer review by panel of experts.

C.12 Does this conclude your testimony?

Yes, it does. In Sections D and E, I have also provided a list of references and supplemental references.

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