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Compilation of Key Documents Pertaining to the

Structural Evaluation of Seabrook Nuclear Power Plant

By

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For

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Newburyport, MA

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Note

This documents contains the major submittal by C-10 to the Nuclear Regulatory Commission (NRC) in connection with the contested license renewal of *Seabrook Station Unit No. 1 Issuance of Amendment No. 159 Re: Methodology for Analysis of Seismic Category I Structures With Concrete Affected by Alkali-Silica Reaction (CAC NO. MF8260; EPID L-2016-LLA-0007)*[1] that is known to suffer from Alkali Aggregate Reaction[2] Additional related documents can be found here.

It is first prefaced by the C-10 Chair of the LAR Task Group (Mr. Chris Nord). Then the credentials of C-10's expert (Dr. Victor Saouma) are highlighted, followed by a summary of the major concerns about the license amendment request (LAR) shown in the appendix.

[1] Included in Appendix **??**

[2] Also known as Alkali Silica Reaction (ASR).

Preface

Those two points (specially the second) is very minimal in the big picture. Suggest that we reman general.

May I suggest with a 2 lines intro as to who is C-10

In their License Amendment Request (LAR) submitted to the Nuclear Regulatory Commission in 2016, NextEra Energy, LLC—owners of the Seabrook atomic power plant—made the following two assertions on consecutive pages:

> *"The large-scale test programs undertaken by NextEra provided data on the limit states that were essential for evaluating seismic Category I structures at Seabrook Station. . . . The results were used to assess the structural limit states and to inform the assessment of design considerations."*
>
> *". . . . the number of available test specimens and the nature of the testing prohibited testing out to ASR levels where there was a clear change in limit state capacity." (ML 16216A240, License Amendment Request 16-03, NextEra Energy—Seabrook, 8/1/2016, 3.2.1: "Structural Limit States")*

This is much better, and no need to be too specific in here, this is done in the subsequent chapters

These two seemingly contradictory statements exemplify the kind of "red flag" warning to members of the C-10 Research and Education Foundation which we found within the LAR. It seemed to us that NextEra was trying to create a methodology for 'monitoring' the progression of Alkali-Silica Reaction (ASR), which could put the best possible "face" on the diminishing durability of their New Hampshire Seacoast reactor—and which therefore might not provide adequate, real protection for those of us living within the reactor community . After all, how can one 'assess limit states" (failure thresholds) unless one tests to failure for different levels of ASR invasion? How can relatively small concrete beams (compared to the massive size of a Containment Enclosure Building), purpose-formed in Texas with fast-acting ASR, be considered "representative" of thirty-plus-year-old concrete structures, standing in a salt marsh in northern New England? Can a test of the expansion of the surface of the concrete possibly tell us the relative health of the interior of a vast structure, upon which the entire surrounding population must rely for their health and safety?

These were some of the questions which we, as laypersons with no professional or academic background in the relevant sciences, had to confront. C-10 had monitored the ASR problem at Seabrook since 2009, when the NRC acknowledged Seabrook's ASR as the first known instance among U.S. commercial reactors. Subsequently, we received guidance and expertise from Dr. Paul Brown, Professor Emeritus in Ceramic Science and Engineering at Pennsylvania State University, in two interventions before the Nuclear Regulatory Commission, on the ASR issue: in 2014, a rule-making petition for inclusion of ASR testing and monitoring standards; and in 2015, an emergency enforcement "2.206" petition on the same issue.

CanI have the url of these?

NextEra's LAR caused C-10 great concern for many reasons; but chief among them was lack of independent review by qualified scientists. The methodologies described within the LAR comprised a new, and untried beyond the laboratory setting, system for ASR 'management;' and the reactor communities within Seabrook's Ingestion Pathway would be unwitting partners to the experiment of using this methodology in the field for the first time—without peer review. We also knew that by 2012, one NRC scientist had quantified the deterioration of safety-related structures at Seabrook at a 22% reduction in compressive strength (Dr. Abdul Sheik, Senior Structural Engineer for the Office of Nuclear Regulation, before the Advisory Committee on Reactor Safeguards, 7/10/12).

Any document to backup the 22%?

Dr. Brown made it clear to us that, in order to challenge the LAR, we would need the expertise of a concrete materials and structural analysis researcher, most likely a University researcher ~~scientist~~.

Because we faced a deadline, in 2017, for submitting contentions by April 10th in order to challenge the LAR, we elected to use the published reports from Dr. Brown to underpin our contentions, knowing that if any of our contentions were accepted, we would need to find a preperly qualified researcher in both concrete ASR and complex structural analysis ~~concrete structural scientist~~ who would be willing to provide the relevant expertise. And, whereas we had represented ourselves *pro se* in the previous petitions before the NRC, it became obvious that we would need to secure legal counsel in order to be properly represented before the Atomic Safety and Licensing Board (ASLB).

Dr. Victor Saouma, Professor of Civil Engineering, University of Colorado at Boulder, consented in 2018 to work as pro-bono expert witness—and has undertaken this role as an academician's ethical obligation, to provide an independent, truthful, and expert review of the LAR (his qualifications follow, at the outset of his testimony). Ms. Diane Curran, Attorney with Harmon, Curran, Spielberg, & Eisenberg, LLP, and a veteran litigator before the NRC who was known by some of C-10's board, accepted our request to join at minimum xxx at the end of 2018, in order to work with Dr. Saouma in preparing our case, and to represent us before the ASLB in formal hearings in September, 2019.

Christopher Nord
C-10 Board Member
Chairman, C-10 LAR Task Group

About the Author

Victor E. Saouma has 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.

For the past four years, he has 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. In that capacity he is the editor of a RILEM report with over 450 pages, and 30 contributors among the top researchers on the related topic of ASR.

He is a past President (and Fellow) of the International Association of Fracture Mechanics for Concrete and Concrete Structures (and hence he is quite familiar with issues pertaining to cracking in concrete).

He has 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.

He was a key contributor to EPRI's report on ASR, Modeling Existing Concrete Containment Structures; Lessons Learned (**eprilessons**).

Other areas of research has included: theoretical, numerical and experimental fracture mechanics, chloride diffusion in concrete, real time hybrid simulation for the dynamic analysis and testing of reinforced concrete frames, centrifuge testing of dams mounted on a shake table.

His international collaboration includes France, Spain, Switzerland, Italy and Japan.

In addition to his training and experience as a scientist, he is also a trained and experienced civil engineer. Most of his research funding has been from sponsors seeking advanced scientific based solutions to practical engineering problems.

He has taught linear and nonlinear structural analyses reinforced and advanced reinforced concrete design. Therefore, he is familiar with and able to evaluate NextEra's engineering-based approach to the problem at ASR at Seabrook.

In studying ASR over many decades, he has 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 his research and publications, and his leadership in an international committee addressing ASR with some of the best researchers in the world, he has acquired a global understanding of the problem that position him to opine with confidence on the adequacy of the work done at Seabrook.

Executive Summary

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Chapter 1

Expert Report; February, 2019

1.1 Introduction and Executive Summary

This report presents my evaluation of the large-scale testing of concrete specimens at the Ferguson Structural Engineering Laboratory (FSEL) at the University of Texas for the purpose of evaluating the effects of Alkali-Silica reaction at the Seabrook nuclear power plant on the ability of the containment to withstand a design-basis earthquake. I have also reviewed the finite element assessment of Seabrook conducted by SGH, a consultant to NextEra Energy Seabrook, LLC.

The FSEL test program suffers from multiple flaws that cannot provide a solid basis for the author's far-reaching conclusions. Three major concerns are noted:

First, as discussed in Section 1.2.3, concrete used in the FSEL tests is not representative of the concrete at Seabrook. This lack of representativeness cuts across virtually every level of the test, including characteristics of the materials tested, test conditions, and assumptions about the behavior of concrete under finite element simulations.

Second, as discussed in Section 1.3.3, the Shear tests do not have the proper boundary conditions. They are also limited to out-of-plane shear, and some large unexplained cracks may have corrupted the test results. In addition, there is no evidence that the limit state (i.e. failure shear force) was captured and thus there is no evidence that shear failure was indeed captured as claimed.

Finally, as discussed in Section 1.4.3, the crack index measurements relied on by the author cannot provide a reliable assessment of the *in-situ* ASR expansion, because a crack index measured on the surface (where the concrete is relatively dry) is not representative of what is happening inside the specimen where the relative humidity (essential for ASR) is much higher.

Of equal – if not greater – concern is the finite element assessment of Seabrook conducted by Simpson, Gumpertz, & Heger (SGH). The numerical technique followed by SGH is a deterministic, linear and simplistic method that is used for the design of new structures. It is very regretful that SGH did not employ in addition to their minimalist analysis the probabilistic risk assessment method pioneered by the NRC. Whereeas this would have required a nonlinear static and seismic analyses, SGH could have obtained a much more accurate assessment commensurate with the needs for such a critical structure. This probabilistic method was pioneered by the NRC and is well-accepted as a useful tool for analyzing the complex interactions of phenomena in nuclear safety analyses. My own independent research, conducted on behalf of the NRC between 2014 and 2017, confirms that probabilistic analysis of ASR yields more credible results than the type of linear and deterministic analysis used by SGH.

The test program at FSEL was executed by a researcher well versed with large-scale testing and reinforced concrete in general. However, his prior exposure to ASR seems to have been limited to a past test program for the Texas Department of Transportation (testing large scale beams with ASR) in collaboration with Prof. Folliard, an internationally known and respected researcher on the matter of ASR. But Dr. Folliard did not participate in the FSEL test program.

In summary, it is my professional opinion that both the FSEL test program and the SGH finite element analysis 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.

Finally, as a researcher and a concerned citizen, it is disturbing that when the NRC was for the first time confronted with such a complex issue it has not subjected the various studies to an independent review panel of international experts. While NRC has stated that it conducted an internal peer review, by its own terms an internal peer review is not independent.

1.1.1 Disclosure

I was the recipient of NRC Grant No: NRC-HQ-60-14-G-0010 titled *Experimental and Numerical Investigation of Alkali Silica Reaction in Nuclear Reactors* from September 30, 2014 to Dec. 30, 2017, and have conducted this evaluation *pro-bono*.

Other qualifications described in separate document.

1.1.2 How to Read this Document

This document describes some of the serious concerns raised by a thorough review of the documents pertaining to NextEra'sinvestigation of the safety implications of Alkali-Silica Reaction (ASR) at the Seabrook nuclear power plant. The document is written to be understood by a layperson, and equally important, it is written with sufficient technical explanation and documentation to provide a technical reviewer with an adequate basis to evaluate my concerns. My report is broken into five sections. In addition to this Introduction (Section 1), Sections 2 through 5 address my concerns regarding the deficiencies of the investigation. Each section begins with, then proceeds with a Background section, and then a critique. To facilitate reading, each Section is broken into three subsections:

- The first subsection, "Relevant Quotes" presents relevant quotations extracted from documents submitted by NextEra (or occasionally other appropriate sources). These quotations set forth NextEra's goals or criteria for a particular test or analysis.
- The second subsection, "Background" enumerates facts that should be kept in mind in the context of my evaluation. This subsection includes testing or analytical requirements that must be satisfied to produce reliable results.
- The third subsection presents my critique of NextEra's data and/or methodology. The critique section will address the pertinent parts of NextEra's reports, and reference those requirements in the Background section that are violated.

The terms *Alkali Silica Reaction* (ASR) and *Alkali Aggregate Reaction* (AAR) are used interchangeably.

All the citation or text in blue are hyperlinks to the corresponding citation, figure, or warning label.

1.2 Concrete

1.2.1 Relevant Quotes

Representativeness *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-Tests-2**

ASR Levels *To make test specimens as representative of Seabrook structures as possible, "testers used" 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-LAR** (pg. 24).

> *Presence 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.* **MPR-Tests-2** (pg. 2-7).

Mechanical Properties *Dr. Bayrak ... stated that the concrete mixture will be sufficiently reactive to obtain the necessary data in a timely manner, and will develop mechanical properties that are representative of Seabrook structures.* **ML121220109** (pg. 5)

1.2.2 Background

Following are some important technical points that should be kept in mind when reading the subsequent subsection criticizing NextEra's reports.

1. Concrete is a delicate dosage of cement, aggregates (about 3/4" max) , sand and water designed to meet specific criteria. Even a minor alteration of one of the ingredients will cause concrete response to change greatly.
2. ASR can be roughly classified into two types: early-expansive ASR and late-expansive ASR (**katayama1997petrography**). Only a proper thin section petrographic analysis can differentiate between the two (**sims2003rilem**).
3. Different kinds of reactive aggregates or sand will cause different types of gel. The calcium content of the gel (**thomas1998role**) is known to be critical in characterizing the ASR expansion.
4. For ASR to occur, three conditions must be met: a) **Aggregates and sand** should contain too much silica; b) the **cement** is sufficiently alkaline and c) relative humidity in the concrete is higher than 80% (**capra03**). When the reaction occurs, gel is formed, fill up the pores, the interface transition zones, and then the concrete will expand and crack.
5. Sand has a larger surface to volume ratio than aggregates.
6. Reaction starts by affecting the aggregates with the highest surface to volume ratio (sand or fine), aggregates react faster, while coarse aggregates react slower but ultimately yield substantially larger expansion.
7. Designing a concrete mix that is representative (i.e. as similar as possible) to the actual concrete (as in Seabrook) is quite challenging, and requires at a minimum a **petrographic analysis** of the site to assess: whether one is in presence of early or fast expanding aggregates; quantification of the damage Fig. 1.1.

Table 1.1: Stages of ASR in concrete (**katayama17**)

Stage	petrographic features
i	Formation of reaction rim, no cracks
ii	Rimming of ASR sol/gel (halo) in cement paste around the reacted aggregate; deposits of gel within voids near the aggregates, no cracks
iii	Cracking of aggregate filled with ASR gel
iv	Propagation of gel-filled cracks from the reacted aggregate into surrounding cement paste
v	Increase of the crack width and precipitation of ASR gel along cracks into air voids distant from the reacted aggregate
vi	Formation of network of gel-filled cracks connecting the reacted aggregate

8. Should there be cracks within the cement paste, i.e. stages iv), v), and vi), a variety of methods could/should be used to **assess the severity of damage**. These include i) crack width or the crack index (total crack width (mm/m) as described by **leemanndiagnosis** and its percentage (%) by **katayama17**, ii) damage levels as described by **menendez2**, iii) damage rating index (**grattan1992comparison**) (**rivard2005**) (**sanchez2015**).

9. After the concrete simulating ASR is cast, extreme care must be taken in "curing" it by: a) **heating** it to accelerate the reaction that otherwise will take many years; and b) preventing **leaching** of the reactive ingredients from the surface by continuously wetting it with water mixing with sodium hydroxide (NaOH). It is common practice for laboratory ASR tests to soak the specimens with water mixed with sodium hydroxide (usually 1.0 M NaOH) to prevent leaching (dilution of sodium hydroxide from the surface which would reduce the ASR expansion (**lindgaard2013**) (**plusquellec2018determining**). Failure to do so, will cause the absence of ASR close to the surface (but not inside the specimens).

10. The **kinetics** of the reaction (that is the progress of the expansion with time) is extremely important to fully understand as it depends on the relative humidity being at least 80%, and the reaction is faster with an increase in temperature (**larive98**) (**ulm00**).

11. It should be mentioned that the FSEL is hosted by the Dept of Civil Engineering at the University of Texas where one of its faculty, Prof. Foliard is an eminent expert on ASR.

1.2.3 Concrete Mix is not Representative

> The concrete mix tested at FSEL is not representative of Seabrook's concrete in several significant respects related to key characteristics of concrete mixes: aggregate, cement, leaching, and kinetics. Because of these anomalies, the specimens tested at FSEL will not behave in a sufficiently similar fashion to the structures at Seabrook. The characteristics with demonstrable differences or significant unknowns are as follows:

Aggregate As mentioned in Section 1.2.1 only *a portion of the coarse aggregate used for the shear and reinforcement anchorage test specimens was transported by trucks to the laboratory in Texas from a quarry in Maine that is near to the quarry where aggregate was obtained for original construction at Seabrook Station* (**SBK-PROP00013490**), this statement raises some serious concerns:

1. What is meant by *portion* i.e., what is the percentage of reactive aggregate, and how was the reactive aggregate distributed among the various shear specimens?
2. There can be substantial difference in silica content within the same quarry (which is why not all the concrete at Seabrook is affected by ASR), and reactive (or non-reactive) aggregates may be produced from geological pockets. Fig. 1.1 illustrate the foundation of high voltage transmission tower in Japan where despite much scrutiny, only one portion is reactive. All the aggregates came from the same quarry.
3. Fine aggregates (sand) was not the same as Seabrook and this is problematic as we know that sand would react first, but will eventually yield smaller expansion than the one induced by reactive aggregates. "Texas fine" is a well known reactive sand, was it used? if so how representative is it of the sand used at Seabrook?

Cement Alkalinity (measured by sodium hydroxide equivalent) plays a big role in stimulating the alkali-silica reaction. There is no indication that the FSEL test used cement with the same alkalinity as the cement at Seabrook. Nor is it clear whether if additional alkaline was used to dope the concrete mix (it is common to provide $\approx 1.4\%$ Na_2O_{eq} through cement or NaOH to accelerate the reaction).

(a) Location of the zone affected by AAR (b) Close-up

Figure 1.1: Heavily reinforced transmission tower foundation partially affected by AAR (**saouma2014AARBook**)

Leaching After casting, and prior to testing, specimens were soaked with water, and no measure to prevent leaching was reported. As a result ASR on the surface is very likely to have been reduced.

Kinetics The kinetics of the reactions in the FSEL are not representative of the one occurring at Seabrook.

1. Per various documents, there are essentially three stated objectives: 1) achieve an expansion consistent with *current* levels of expansions; 2) levels that could be observed in the *future*; and 3) develop a concrete mix that is sufficiently reactive in a timely manner.
2. In order to scientifically duplicate *currently observed ASR at Seabrook* it is necessary to have at a minimum: a) thin section petrographic analysis to estimate the damage **sims2003rilem** (**katayama17**); or 2) other carefully performed expansion tests (**merz13**). This was not reported.
3. Designing a concrete mix that will result in an expansion that fulfills *simultaneously* the three requirements: consistent with present and future ones at Seabrook, while expanding sufficiently rapidly to obtain data in a *timely* manner while developing mechanical properties (specifically: compressive strength f'_c, tensile strength f'_t, Elastic modulus E, fracture energy G_F, creep coefficient Φ), or the calcium content of the gel (**thomas1998role**) known to be critical in characterizing the ASR expansion, is impossible.
4. From the report, it is not clear what measures were taken to accelerate the reaction in the laboratory[1], yet a through-thickness expansion as high as [redacted]% is reported (**SBK-PROP00013490**). One has to assume that this corresponds to the *future* expansion (though this is a dangerously large strain value).
5. Achieving [redacted]% expansion in less than a year within an unheated room is very puzzling, and calls into question the concrete mix representativeness.
6. For the tests to be representative of current and future expansions, at a minimum the specimens must have been kept in a "high" temperature environment to accelerate the tests. Yet, there is no indication in the report of such curing conditions.
7. An attempt was made to elucidate this:
   - It can be reasonably assumed that the storage temperature at FSEL was 30°C.

[1]Surprisingly, the term *temperature* does not appear once in **SBK-PROP00013490** .

- Seabrook's external average temperature at the site is estimated to be 11°C (external face of NCVS), the internal temperature is in turn estimated to be 25°C. Hence, an average mean yearly temperature of (25+11)/2 = 18°C can be assumed.
- From Arrhenius law (**ulm00**), one can draw the normalized expansion curve at Seabrook, and the one at the FSEL, Fig. 1.2. Making a reasonable assumption that the AAR expansion on site has reached 0.15%), it would then take about 7 years at the FSEL to reach this value.

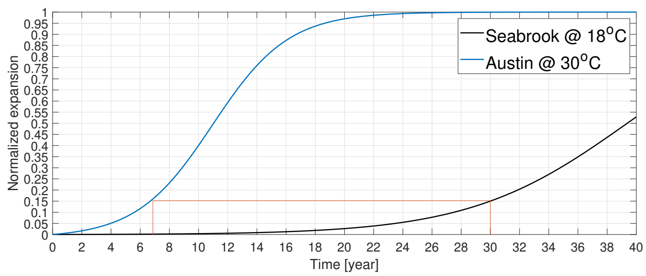


Figure 1.2: Normalized expansion at Seabrook site and FSEL

8. Finally, it should be noted that a volumetric expansion of even 0.7% when properly used in a finite element simulation will result in substantial degradation (**saouma-nrc-report-fea**).

1.3 Structural Shear Tests

1.3.1 Relevant Quotes

Representativeness of specimen: *Application of the results of the FSEL test programs requires that the test specimens be representative of reinforced concrete at Seabrook Station* **MPR-Tests-2**

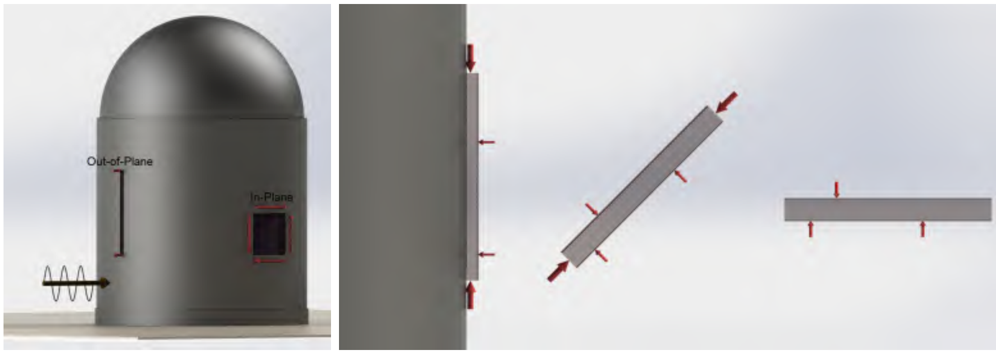
Confinement and Boundary Conditions: *The presence of confinement is a central factor for the effect of ASR on structural performance. Reinforcing steel, loads on the concrete structure (e.g., dead weight), and the configuration of the structure (i.e., restraint offered by the structural layout) may provide confinement that restrains in-situ expansions due to ASR and limits the resulting cracking in concrete.* **MPR-4288** (Sect. 2.2)

Limit States: *... objectives of NextEra's evaluation of the effects of ASR on Seabrook structures with regard to: (a) load carrying capacity for critical structural limit states and other design considerations to demonstrate that Seabrook structures with ASR meet the strength requirements of ACI 318-71 (the design code of record)* **NRC-LAR** (Sect. 3.2)

1.3.2 Background

Following are some important technical points that should be kept in mind when reading the subsequent subsection criticizing NextEra's reports.

1. What is at stake is the response of the NCVS when subjected to a **lateral seismic load** (ground motion). This will subject it to shear forces that must be resisted.
2. The (cylindrical) container will resist shear through two mechanisms: a) **Out of plane** shear at the azimuth along the earthquake; b) **in plane** shear at 90 degrees; and c) a combination of the two in between, Fig. 1.3(a).
3. In both cases, relevant concrete tests should subject their specimens to large **normal compressive stresses** caused by the weight of the dome and wall above the lower portion of the cylinder (where shear is highest, Fig. 1.3(b).



(a) Shear resistance to an earthquake in an NCVS

(b) From Seabrook to FSEL; Tests ignored axial forces

Figure 1.3: Representativeness of the FSEL tests

4. Compliance with **aci318** shear provisions implies that one can determine the *ultimate shear strength* of a beam (Similar provisions were already present in the 1971 version of the ACI code).

1.3.3 Shear Tests are not Representative

Shear tests are not representative of the structural concrete in an NCVS.

Limited Applicability Only out of plane shear tests were performed (Fig. 1.3(a)). Even setting aside the questionable applicability of these tests results, they are only applicable in a narrow zone of the NCVS, Fig. 1.3(a).

Boundary Conditions applied on the specimens are erroneous. Whereas the lateral confinement can (to some extent) be replicated by the large number of transversal rebars, the axial force exercised by the self weight (i.e. weight of the structure above the point under consideration) of the containment structure, Fig. 1.3(b), is not modeled above. Given that the ASR expansion will induce (structurally) some beneficial confining forces, those may be negligible compared to those exercised by the (non-modeled) axial ones.

No Evidence of Shear Strength The report provides no evidence that the tests reached the limit state load. Since the tests were conducted under load and not under, displacement control (closed loop), there should have been an abrupt drop in the load when the limit state was reached (**SBK-PROP00013490**). The load displacement curve instead plateaus, Fig. 1.4(b). This implies that some steel is yielding, yet there should not have been shear reinforcement at the location of shear failure (**SBK-PROP00013490**) other than at the supports. Incidentally this figure mixes up *plan* with *elevation* (not consistent with Table 3-1!).

Structural *vs.* **Material Tests** What is being tested is a beam analogous to a segment extracted from the NCVS. At best it can be used to validate an analysis of a beam and not the other way around (to provide material properties for a finite element analysis of a beam or a NCVS).

Observed 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. Figure 4-2 is a photograph, Fig. 2, showing the large crack in one of the beam specimens,*(**MPR-4288**). This unanticipated large crack on specimen edge (**MPR-Tests-2**) is of major concern. Though mentioned in the report, there is no explanation as to what may have caused it, nor the possible impact it could have on the test results.

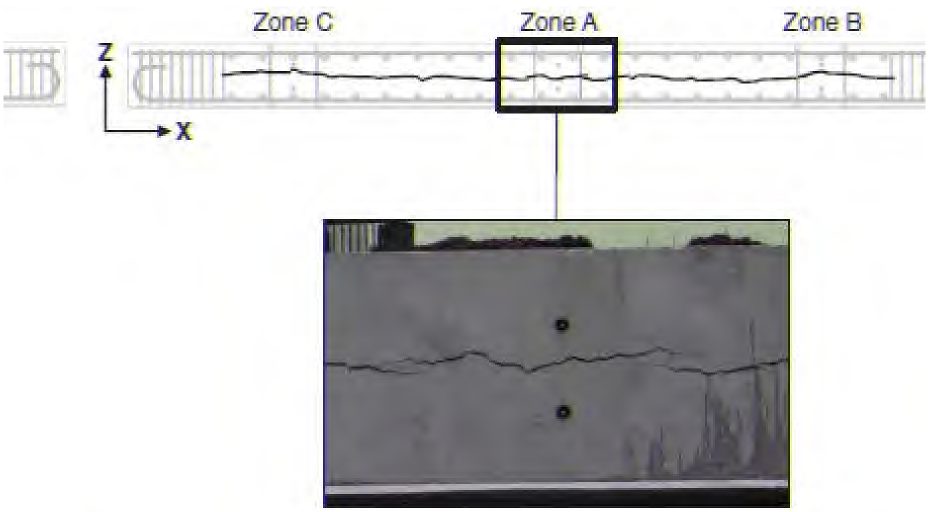


FIGURE 8 Mid-depth surface crack on x–z specimen face.

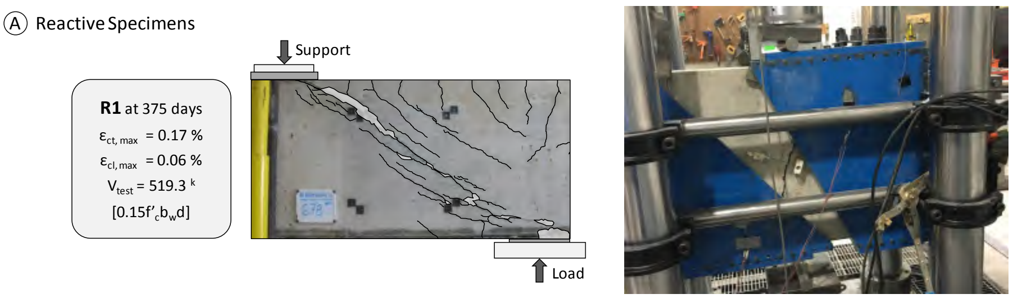
(a) Observed and unexplained structural crack, (**wald2017expansion**)

[redacted]

(b) Reported load displacement curves of selected specimens (**MPR-4288**)

Figure 1.4: Some problematic observations

No Images of Specimens Every test program must show pictures of the tested specimens before **and** at failure (as in Fig. 1.5). Understandably, shear cracks may be clouded by the presence of ASR induced cracks, however those should be clearly identifiable. In this case there is not a single picture with evidence of a shear failure as professed. This is of great concern.



(a) Example of reported shear cracks in a reinforced concrete beam suffereing from ASR **deschenes2009asr**

(b) Diagonal shear crack at failure failure (**saouma-nrc-report-shear**)

Figure 1.5: Examples of reported pictures of shear cracks by other researchers

1.4 Monitoring Crack Index

1.4.1 Relevant Quotes

Applicability Since Time of Construction: *An important advantage of the [Cumulative Crack Index] CCI methodology for Seabrook Station is that results can be used to approximate total expansion in the in-plane directions since the time of original construction.* **MPR-Tests-2** (§4.2.2).

1.4.2 Background

1. The water/cement ratio in concrete mixes is higher than the amount due for hydration of the cement so as to facilitate fluidity of the concrete during placement. Hence, surface relative humidity is most often (if not nearly always) lower than inside (**fournier2000alkali**) (**jensen2003relative**).

2. For ASR to proceed, there has to be a relative humidity of at least 80% (**capra03**).

3. Hence, surface in-plane measurements of ASR expansion, not only cannot reflect the wall expansion, but they will yield *conservatively* low values. The ASR expansion inside the wall is substantially higher than on the dried wall surface.

1.4.3 Cumulative Crack Indexes are Unconservative

CCI cannot be used to assess in-plane expansion.

Based on the above, CCI are dangerously unconservative to assess in-plane ASR deformation.

Crack index measured on the surface are not representative of what is happening inside the specimen (beyond $\simeq$ 2-3 inches).

Its use was justified in an earlier document by its adoption by the Federal highway administration (**fhwa10**). However this FHWA report 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 *assessment of the current condition, i.e., determination of the degree of expansion/damage reached to date, and of the trend for future deterioration of the concrete undergoing ASR expansion.*.

1.5 Consequential Finite Element Analyses

1.5.1 Relevant Quotes

Margin of Failure *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.* **american2016seismic** (American Society of Civil Engineers 4-16, 2016)

1.5.2 Background

1.5.2.1 Probabilistic Analysis

1. 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 (**beckjord1993probabilistic**).

2. The landmark report **wash-1400** cast the foundations for probabilistic risk assessments.

3. Following the 1979 accident at Three Mile Island, it was recommended that PRA be used to complement the traditional deterministic methods of analyzing NPP safety and that probabilistic safety goals be developed for nuclear plants (**kemeny1979report**) and (**nureg1250**).

4. After Fukushima, plants were required to reevaluate the potential impact of external events on their structures (**epri-fukushima**) as in some cases the seismic stressor may have been underestimated (**hardy2015us**).

5. There has been a gradual shift toward probabilistic risk evaluations in recent years. **kennedy1984seismic** 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.

6. It should be emphasized that PRA has a strictly defined meaning within the nuclear community and is an over arching study. In the context of this report the term PRA is limited to the seismic structural analysis of a structure. This is best illustrated by Fig. 1.6 extracted from (**Saouma2019Seismic**) where:

   Aleatory Uncertainties represent variability that cannot be reduced but can be characterized. In this study, is associated with ground motion selection. The uncertainty analysis starts with the hazard curves, site identifications, deaggregation model, and site specific acceleration response spectra. Finally, a set of real ground motions are selected and scaled to be consistent in the aggregate with the design response spectrum (**lapajne1997seismic**; **JayaramLinBaker2011**; **dolvsek2012simplified**; **bradley2017guidance**). Alternatively, one may generate synthetic ground motion or even intensifying acceleration functions (**mashayekhi2018development**; **rezaeian2008stochastic**).

Epistemic Uncertainties represent variability that can be reduced by data collection or experimentation. In our unpublished study, it is associated with material and modeling and is best accomplished through Monte Carlo Simulation (MCS) Family (**VamvatsikosFragiadakis2010**; **HaririSaoumaSensitivity**). The finite element model preparation is included in this block.

Finite Element Simulations is where the preceding two uncertainties are combined and dynamic analyses performed. Again, whereas response spectrum method (RSM) was adequate early on (**ashar2001code**), time history is now strongly recommended. The analyses should be nonlinear so as to capture the various levels of engineering demand parameters (EDP). Different techniques exist for multiple dynamic analyses (**Vamvatsikos2002a**; **JalayerPHD2003**; **jalayer2014bayesian**; **zentner2011numerical**; **huang2017evaluation**).

Post-Processing of the results to extract EDPs, construct capacity functions (**hariri2017single**), and finally compare the results with multiple limit states (LS) functions and drive fragility curves or surfaces **Baker021113EQS025M**; **Porter˙fragilitycombination**; **huang2011probabilistic**; **huang2011probabilistic2**.

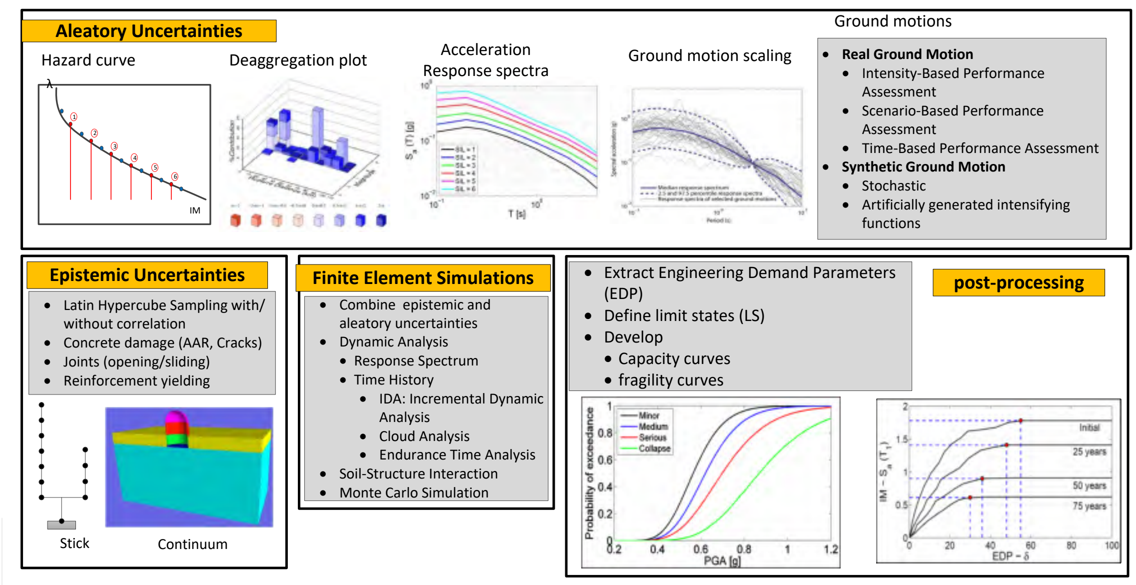


Figure 1.6: Anatomy of a Seismic Probability Risk Assessment

Finally based on my past experience in applying probabilistic, statistical and risk based concepts for the analysis of structures affected by ASR (**lee2006probabilistic**), (**saouma2016effect**), (**saouma2017riskinformed**), (**hariri2017single**), (**shear-wall2018**), (**Saouma2019Seismic**), (**hariri2018random**), (**prob-frame-aar**), (**stoch-fe-aar**), I can conclude that **deterministic analyses should be avoided at all cost as they may provide a false sense of security for the analyzed structure.**

1.5.2.2 Dynamic Analysis

1. Historically, engineers have favored modal analysis in conjunction with the response spectrum method (RSM) as it was not CPU intensive. However, one must recognize that the RSM is a very approximate method which only produces positive values of displacements and member forces which are not in equilibrium; thus demand/capacity ratios have very large errors.

Interestingly, Prof. Wilson is reported to have said: *Ray Clough and I regret we created the approximate response spectrum method for seismic analysis of structures in 1962.... At that time many members of the profession were using the sum of the absolute values of the modal values to estimate the maximum member forces. Ray suggested we use the SRSS method to combine the modal values. However, I am the one who put the approximate method in many dynamic analysis programs which allowed engineers to produce meaningless positive numbers of little or no value... After working with the RSM for over 50 years, I recommend it not be used for seismic analysis* (**wilson-response2**).

2. Nevertheless, linear analysis is typically performed during the design process and does not provide a picture of the actual margin of safety, **nor does it provide enough information to make realistic estimates of seismic risk** (**american2016seismic**). **A true margin of safety** (critical when potential leak may occur as a result of concrete degradation) **can only be achieved through a nonlinear analysis**.

1.5.2.3 Finite element Studies and Supporting Tests

1. Complex structures are analyzed by the finite element method. The structure is essentially "broken" into thousands of individual simple elements (such as hexahedral elements in 3D). We certainly break the structure into its elementary constituents (such as steel, concrete, concrete with ASR, foundation, etc).

2. The method hinges on our ability to characterize the **material** through a so-called constitutive model, that is the nonlinear stress-strain curve in this case. Then an analysis is performed, and ideally validation can be performed (such as verifying that the finite element model could predict the failure of the FSEL tests. The constitutive model is obtained from **material laboratory tests** and never from structural component tests. To determine input material properties in a finite element analysis from structural tests, one must the use perform a system identification analysis (**alves2006system**).

3. Simply put, one should distinguish material from structural tests (**lakes1993materials**).

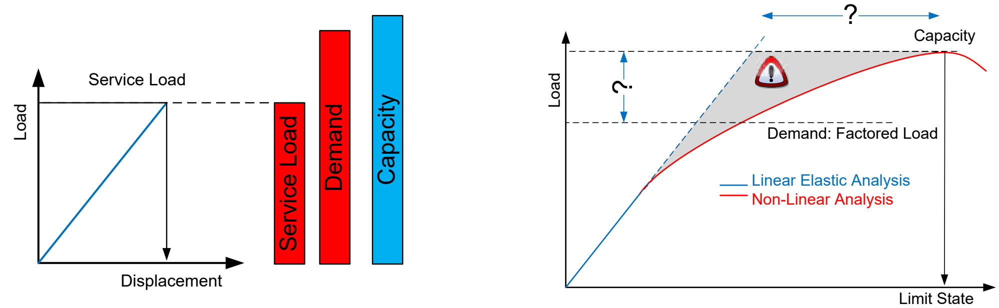
1.5.2.4 Nonlinear Analysis; Design *vs.* Analysis

1. In the context of designing a new structure, one can go along with **aci318** assumption: amplify the load assuming that this will be equivalent to the full "demand", and then compare the demand with the "capacity" using a so called plastic analysis. Verification could then be done element by element, Fig. 1.7(a).

2. However, in the presence of an **existing structure suffering from ASR**, and given what is at stake (public safety ultimately) one cannot, and should not use design procedure (based on linear elastic assessment of the Demand), but use a nonlinear analysis of the entire structure (preferably in the context of a probabilistic investigation), Fig. 1.7(b).

3. This is necessary as strength is not the only criteria, so-called serviceability (unfactored or service loads) is equally if not more important for NCVS. In this context, this will govern the formation of concrete cracks that may constitute a pathway for radioactive gas (in case of core damage, and liner failure). However, cracks cannot be estimated from a linear elastic RSM-based analysis (neither the corresponding displacements), but can only be captured from a nonlinear analysis where the large inelastic displacements can be indeed captured.

4. **nureg6906** provided an excellent set of guidelines for the nonlinear analysis of NCVS; however, it seems to have been seldom consulted by published documents, and is by now mostly obsolete. More recently a short EPRI report (**eprilessons**) addressed concrete degradation, beyond-design basis analysis, forensic analysis of concrete structures, and structural modeling. A comprehensive set of guidelines were submitted to the NRC by the authors (**saouma-nrc-report-fea**).



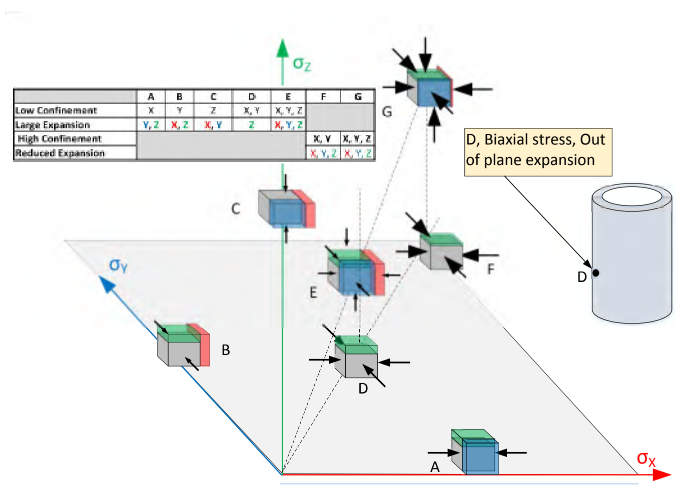
(a) Safety assessment of one elment (b) Fragility Analysis of a structure

Figure 1.7: Concerns with nonlinear analysis

1.5.2.5 AAR Analysis

AAR modeling has to take into account:

1. The spatial variation of the affected zones (the transition zone from one to another may induce sharp discontinuities in stresses and resulting cracks).
2. Compressive stress greater than about 8 MPa will either limit or entirely prevent expansion in the corresponding direction (**Hayward88**). Under complex triaxial state of stress (as in NCVS), expansion will redirect in the other directions (thus inducing an anisotropic expansion) (**multon06**) (**liaudat2018asr**), Fig. 1.8.



	A	B	C	D	E	F	G
Low Confinement	X	Y	Z	X, Y	X, Y, Z		
Large Expansion	Y, Z	X, Z	X, Y	Z	X, Y, Z		
High Confinement						X, Y	X, Y, Z
Reduced Expansion						X, Y, Z	X, Y, Z

Figure 1.8: Stress induced anisotropy during volumetric AAR expansion

3. ASR will reduce the tensile strength and the elastic modulus of concrete (**esposito2016influence**) (**fhwa10**) by as much as 60%. As to the compressive strength, it has long been assumed that it is not affected by ASR; however there is recent evidence to the contrary, (**esposito2016influence**).
4. It is grossly inaccurate to model ASR as a thermal load as it cannot capture the anisotropic expansion, is not cross-correlated with the presence of sufficient relative humidity, cannot capture the kinetic of the reaction. (**saouma2014AARBook**).
5. When performing a complex analysis with new challenges (ASR in this case), a **finite element AAR program must be validated**. **RILEM-Benchmark-AAR** provides a li-

brary of benchmark problems for quick code validation. Such a validation is not uncommon (**merlin-aar**).

6. There are many published papers properly addressing the modeling of ASR in a scientific approach reflecting the state of the art. It was the pioneering work of the French at the *Laboratoire Central des Ponts et Chaussées* (LCPC) that set the ground for most of this work (**larive98**). Subsequently, many researchers have addressed in more details one or more of the far-reaching observation of Larive. They include (and are not limited to) (**bazant00**), (**multon05**), (**sellier2009**), (**Comi09**), (**rodriguez11**), (**pian12**), (**alnaggar2013lattice**), (**vector3**) (**huang2015grizzly**), (**huang15**), (**benftma16**). To the best of my understanding no researchers has so far rejected the premises of (**larive98**) (i.e the impact of stress induced anisotropy, role of temperature and relative humidity, concrete degradation due to ASR.

In summary, ASR will reduce the tensile and shear strength of concrete while increasing it s propensity to larger deformation. This in turn increases the likelihood of cracking and reduces the ability of a structure to resist lateral seismic load.

1.5.2.6 Shear Strength and AAR

1. As a material, concrete shear strength will also degrade as it is intimately related to the tensile strength (**nilsonbook**). Indeed the ACI code (**aci318**) estimates the tensile strength of concrete to be $f'_t = 4\sqrt{f'_c}$, whereas the shear strength to be $v_n = 2\sqrt{f'_c}$ where f'_c is the compressive strength. Then, there is irrefutable evidence that ASR reduces drastically the concrete tensile strength (**fhwa10**), and hence ASR *must* also reduce concrete shear strength.

2. Some tests argue that ASR increase the shear strength of concrete. This is wrong, the statement should read *ASR increases the shear strength of reinforced concrete structural components.*

3. The shear strength of (Materials) concrete is undoubtedly reduced by AAR for very simple set or reasons. From Mohr circle, it suffice to rotate by 45° a concrete element subjected to pure uniaxial tension to obtain a specimen under shear. Indeed the ACI code (**aci318**) estimates the tensile strength of concrete to be $f'_t = 4\sqrt{f'_c}$, whereas the shear strength to be $v_n = 2\sqrt{f'_c}$ where f'_c is the compressive strength. Then, there is irrefutable evidence that ASR reduces drastically the concrete tensile strength (**fhwa10**), and hence ASR *must* also reduce concrete shear strength, (**chana1992structural**), (**chana1992bstructural**), (**ahmed1998state**), (**schmidt2014novel**), (**saouma-nrc-report-shear**).

4. The only reason tests argue that ASR increase the shear strength of concrete, is simply because the statement should read *ASR increases the shear strength of reinforced concrete structural components.*

5. All tests that have shown increase of shear strength were performed on reinforced concrete structural component (beams) and never on plain concrete.

6. It is very important to make the distinction between testing a structural component (such as a reinforced concrete beam) *vs.* testing concrete material.

7. Results of a structural test, cannot be directly applicable in a the finite element analysis of **another** structure.

1.5.3 Finite Element Simulations are not Representative

the Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction is very simplistic with countless erroneous assumptions, modeling simplifications.

The methodology described in **MPR-FEA-3** is flawed. Its subsequent implementation in (**SGH-FEA-4**) is therefore critically reviewed:

1. **pg. 1** *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.

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 **pg. 15** *The strains due to ASR expansion simulated by the finite element model (FEM) reasonably approximate crack index measurements .*

   As previously mentioned, the CI readings are notoriously unreliable as the concrete surface has dried and as such has reduce ASR expansion. If correlation is made with the out of plane expansion measurements device (from Geokon), it is surprising that a shell element (that does not capture the thickness effect, and the RH distribution) could could reliably duplicate the field measurements of ASR expansion.

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*

   On what basis is it assumed that future expansion will increase by 20%? How is "future" defined? At the very least expansion tests should be performed to have *an idea* of what the future expansion will be.

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

   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.

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.*

   To the best of my knowledge, no researchers has succeeded this feat of making such a grandiose claim.

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 the ACI code (**aci318**) 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.*

Incorrect, it is stated separately that *a portion of the coarse aggregate used for the shear and reinforcement anchorage test specimens was transported by trucks to the laboratory in Texas from a quarry in Maine that is near to the quarry where aggregate was obtained for original construction at Seabrook Station* (**SBK-PROP00013490**). Furthermore, the source of the sand is unknown. In other words one cannot reasonably assume that the tests had the same concrete as Seabrook.

8. **pg. 23, JA07** *ASR cracking typically has a map pattern, which is generally less apparent at the springline elevation than other ASR monitoring locations.*

   Indeed one would have map cracking in unreinforced concrete. In reinforced concrete, cracks will have a preferential direction (along the reinforcement) and can be readily identified, Fig. 1.9.

Figure 1.9: Cracking in reinforced concrete beam and column caused by ASR (**fhwa06**)

9. **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.*

   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 a material testing (where it does not) needed for a finite element analysis. Furthermore, Strains in the linear elastic analysis (performed) are grossly underestimated.

10. **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.

11. **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, Fig. 1.10(a).

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

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

13. **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.*

    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.

14. **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.

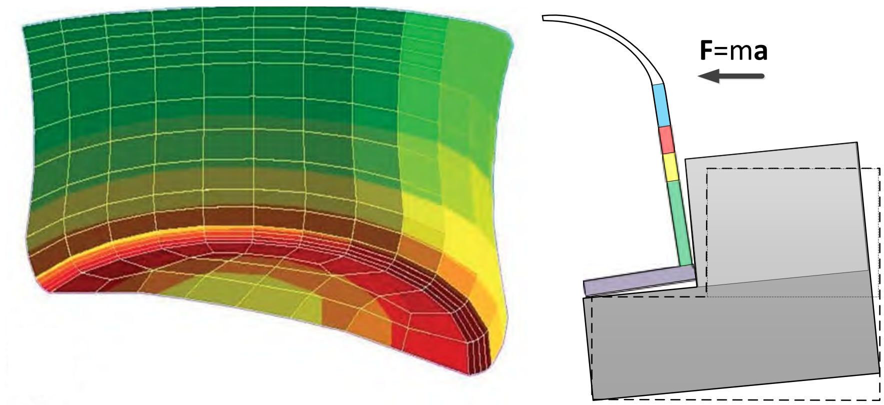
15. **pg. 44** *Response spectra analysis was performed using a simplified "stick" model. For lateral analyses, the model was fully fixed below EI. 0 ft. For vertical analyses, the model was fixed at the base at El. (-)30 ft.*

    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. The model cannot capture the seismic contact between the (ASR induced wall expanded wall) with the adjacent soil unless joint elements are inserted, Fig. 1.10(b).

    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 (**ashar2001code**).

16. **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.



(a) Anticipated deformation of the base mat (note upward deflection) (b) Potential uplift modeled by joint elements

Figure 1.10: Some ignored considerations in the analysis that may severely impact results (**Saouma2019Seismic**)

One should have examined indeed element by element and assess strength through an 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.

17. **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.*

    No basis for such a statement.

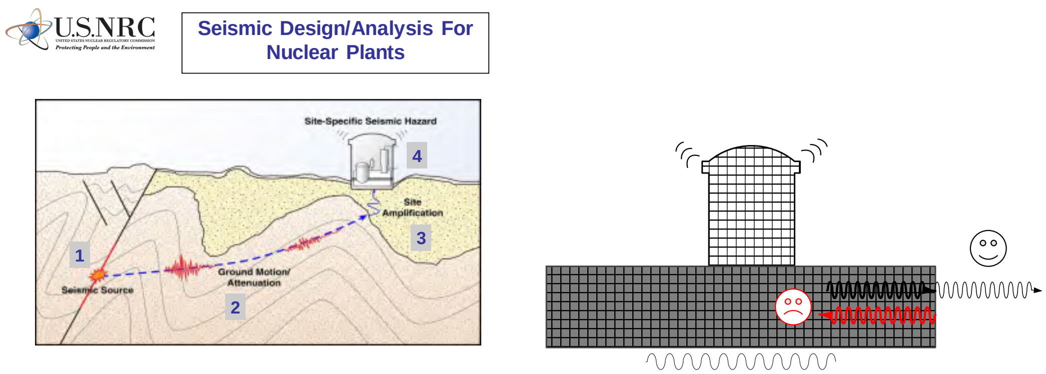
18. **pg. F-3** *Cracked section properties do not affect the global seismic response of the CEB. This assumption is justified because the global response of the CEB to seismic motion primarily causes in-plane shear and overturning stresses; both are resisted by the membrane stiffnesses of the CEB wall that are not impacted by cracking.*

    The global seismic response will (in this case) be primarily affected by the reduced elastic modulus (ignored).

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

    Because of the linear elastic analysis, the strains are grossly underestimated.

20. Impact of soil through proper deconvolution or soil structure interaction is not accounted for as required by (**american2016seismic**) (for the later).



(a) Seismic Attenuation/Amplification (**pires**)

(b) Need for soil-structure interaction in seismic analysis (**saouma-nrc-report-fea**)

Figure 1.11: Not modeled impact of soil in the seismic analysis

1.6 Summary and Recommendations

1. Crack Index measurement are extremely unreliable indicator of the true ASR expansion occurring inside the wall and should be avoided at all cost as they can result in false-negative results.

2. Though not addressed in the review, the techniques to measure the out of plane ASR deformation is an interesting one provided they are inserted well inside the wall and not on the surface.

3. The concrete used in the FSEL is indeed an expanding one, but most certainly is not similar to the one at Seabrook.

4. I have not commented on the anchorage tests as I do not feel sufficiently qualified to address them.

5. Shear test results are not entirely credible and irrelevant to the subsequent finite element simulations. They could have been easily anticipated, and are not in contradiction with those reported in the literature.

6. Perform diagnostic and prognosis laboratory tests. The former through a detailed petrographic analysis to quantify the crack damage index (not to be confused with damage index), presence of gel, type of gel and others. The later through a series of carefully performed expansion tests of concrete samples from Seabrook.

7. The methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction is flawed and dangerous if adopted.

8. Whatever "conservativeness" is presumed in the calculations (and not all are indeed valid) is completely overshadowed by the many other unsubstantiated assumptions, and thus can not be relied upon.

9. The Finite element studies constitute a first step, but need to be complemented by an additional battery of analyses before any major conclusion can be drawn. More specifically

    (a) Use a mesh with solid 3D elements and do not use shell or stick meshes.

(b) Make sure that the proper boundary conditions are modeled (allow for separation of the mat from the foundation and sliding of the wall with respect to the adjacent soil (specially for seismic analysis.

(c) Properly model the ASR to account for stress induced anisotropy, kinetics of the reaction (from the laboratory tests), effect of temperature and relative humidity. Finite element code should first be validated with some simple problems to be credible.

(d) Replace the response spectrum method with an implicit nonlinear time history analysis. This method is available in most programs.

(e) Determine the presence (or absence) of cracks.

10. Determine the margin of safety (i.e. provide a fragility surface in terms of ASR and ground motion excitation.

Chapter 2

Pre-filed Testimony; June, 2019

2.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* (**saouma2014AARBook**).

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 (**saouma13b**).

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**)."

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") (**MPR-FEA-3**) **(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) (**SGH-MPR-FEA-1**) (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* (**SGH-Factors**) (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. (**SGH-FEA-4-conf**) (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**) (**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).

2.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 What legal or industry standards are applicable to ASR?

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 **fhwa10**.

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 2.1. 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.

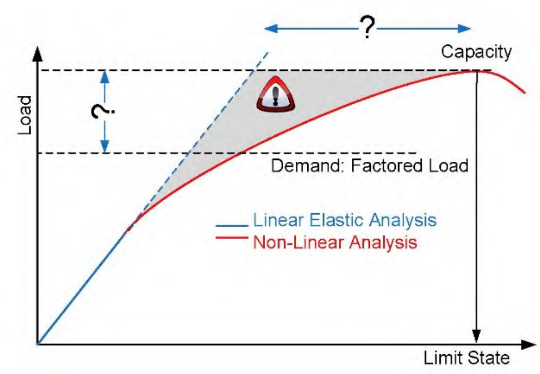


Figure 2.1: Design vs Analysis

2.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, (**SBK-PROP00013490**), 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." (**SBK-PROP00013490**) 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.

In fact, I have confirmed that the concrete was [redacted]. (**SBK-PROP00013490**) page 3-1 – 3-2 **(Exh. INT021) (Proprietary).**

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

- It is well established (**poyet07**) 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 (**katayama17**). 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 2.2. 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.

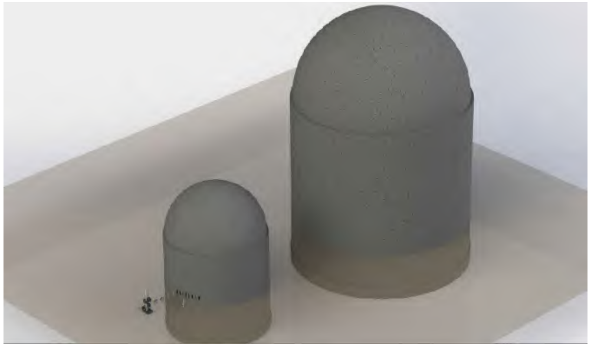


Figure 2.2: Example of a prototype and scaled down model from where a "test specimen" is extracted for laboratory testing.

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

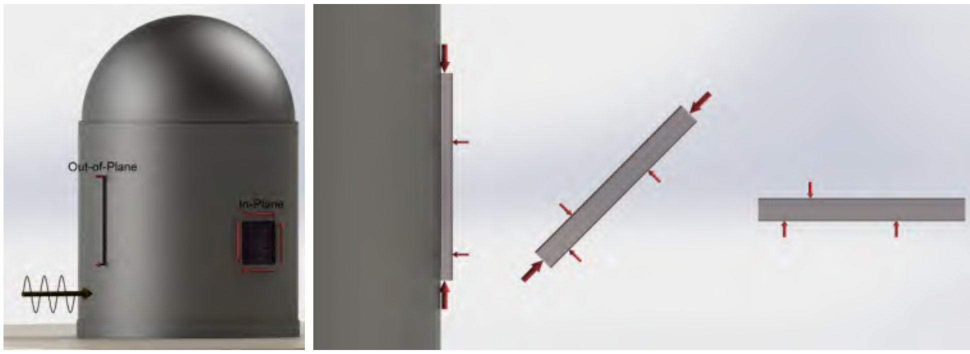
C.2.2.2 Boundary Conditions:

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 2.2:

- 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 2.3(a).
- The beam (shown below in Figure 2.3(b)) was restrained from lateral expansion due to the adjacent concrete. This was accomplished by placing reinforcement as shown in .
- 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.

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

Failure to Address Load Displacement, Cracking



(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 2.3: Boundary Conditions

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 2.5-a.

However, the FSEL test did NOT result in a crack. The reported load displacement curve is as shown below in Figure 2.4.

[redacted]

Figure 2.4: Reported load deflection

As shown, there is barely a "blip" (circled in Figure **??**) 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 2.5-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.

The authors should have included in such an important report pictures of shear cracks, similar to

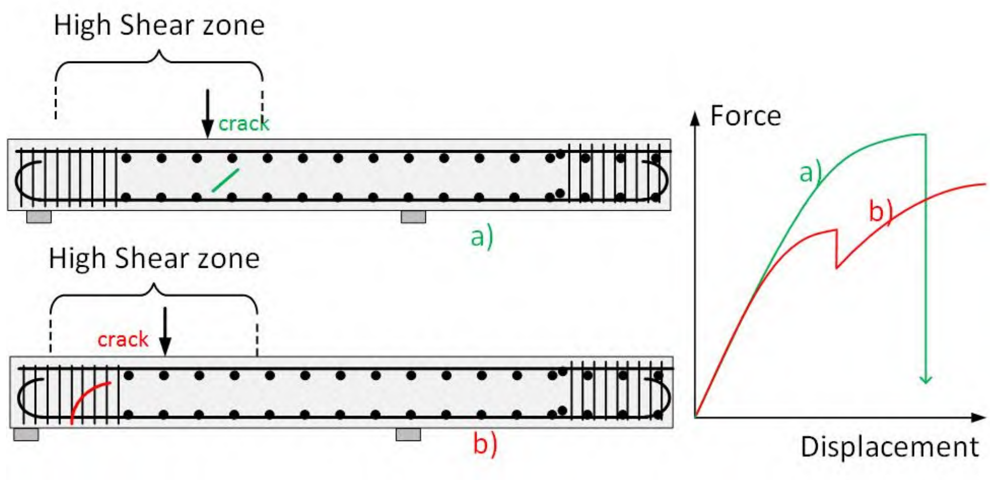


Figure 2.5: "Shear" crack formation locations and corresponding load displacement response; Test probably was intended to have a crack in the top figure (green), but ended up occurring in the location shown below (red).

the one shown in Figure 2.6(a) (based on a test with axial forces shown in Figure 2.6(b)). Surely, they were aware of the importance of such a picture as The FSEL (**deschenes2009asr**) has published one (Fig. 3-10) in a report to the Texas Department of Transportation, Figure 2.7.

C.2.3.2 Cracking:

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

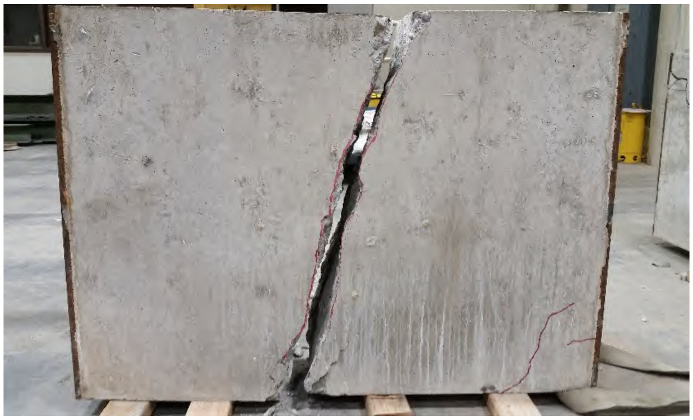
> *As ASR developed in the test specimens, a large crack was noted in the center of the surfaces of the beam that were between the reinforcement mats. This large crack is not representative of expansion behavior of structures at Seabrook Station, which have a network of members that are either cast together or integrally cast with special joint reinforcing details.*
>
> (**SBK-PROP00013490**) p. 4-4 **(Exh. INT019).**

Such an unanticipated crack, shown in Figure 2.8, should be of the utmost concern as it jeopardizes the representativeness of the ensuing test.

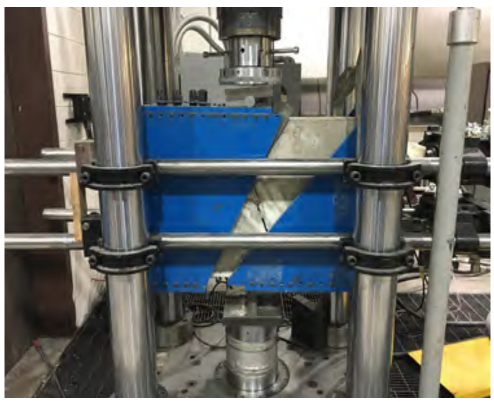
There is a very simple explanation (regretfully, none was provided in the report) as to what most likely has happened. Figure 2.9 provides an illustration. Expansion is restrained along the beam in the X and Y axis, but not in the Z axis. Hence ASR volumetric expansion was nearly entirely channeled into 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 2.10.

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

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



(a) Example of a shear crack



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

Figure 2.6: Example of a shear crack

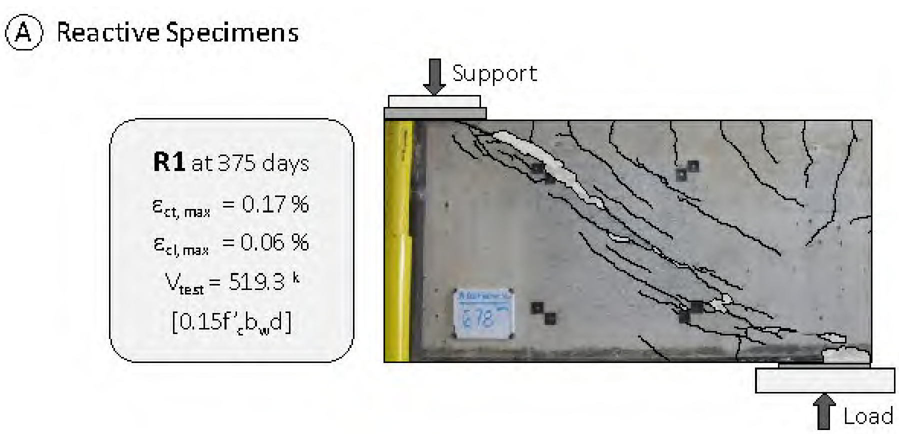


Figure 2.7: Example of shear crack picture reported by **deschenes2009asr**

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

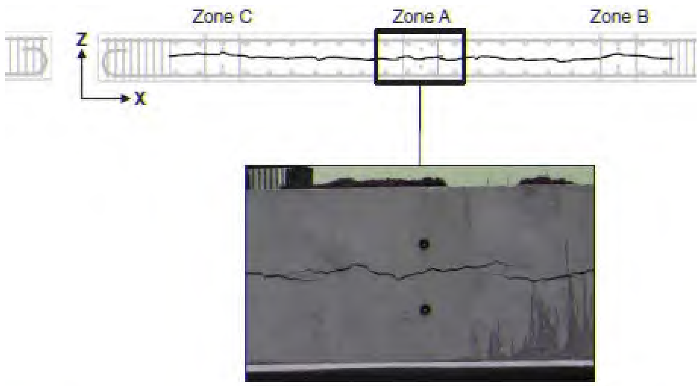


FIGURE 8 Mid-depth surface crack on x–z specimen face.

Figure 2.8: Reported Large Crack from Surface between Reinforcement Mats (**wald2017expansion**)

[redacted]

Figure 2.9: 4.2 in MPR-4273, Rev. 1 (Exh. INT021) (Proprietary))

Figure 2.10: 0 Crystal River delamination (crack between the two reinforcing mats)

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 (**alves2006system**). Simply put, one should distinguish material from structural tests (**lakes1993materials**).

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 (**SGH-Factors**) **(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 2.11.

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 2.11, 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?

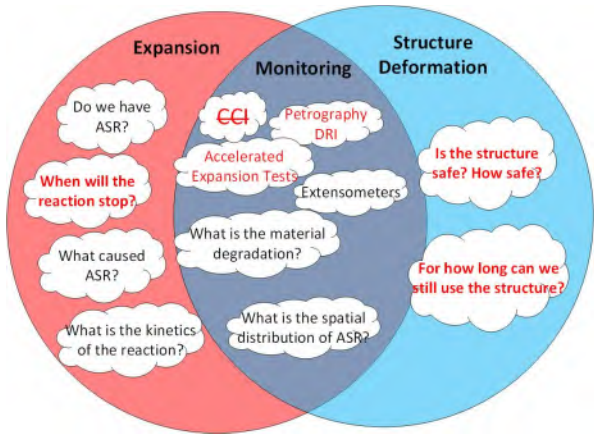


Figure 2.11: Interconnection between AAR and Structural Monitoring

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 2.12 of this FHWA report by **fhwa10** 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.

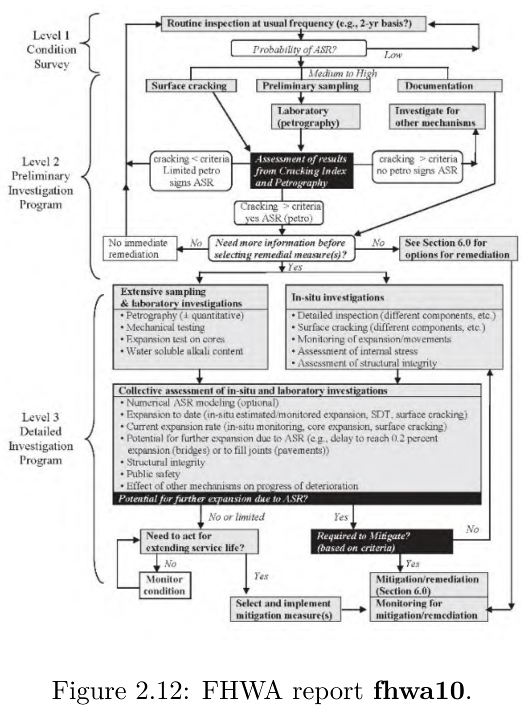


Figure 2.12: FHWA report **fhwa10**.

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 (**stark1987alkali**). 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 2.13.

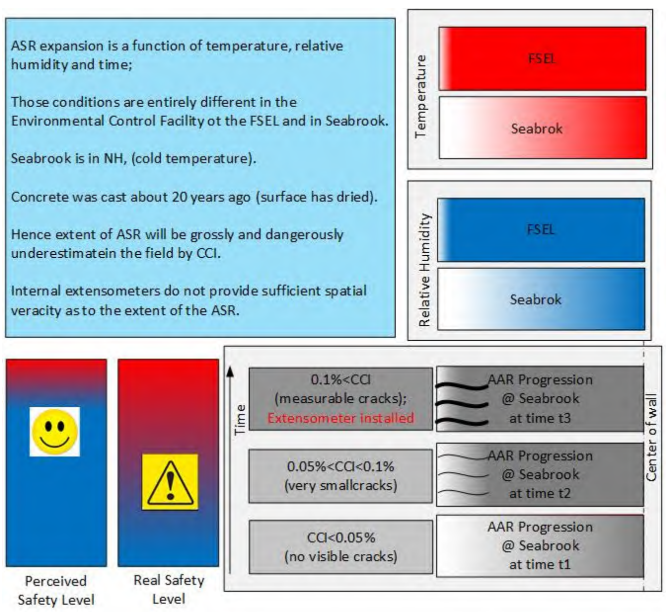


Figure 2.13: Progression of ASR and likelihood of being captured by CCI

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

C.3.2 What other physical in-situ measurements should be employed?

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 2.16(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 2.14

Table 6. Structure Deformation Monitoring Requirements

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

Figure 2.14: Structure deformation monitoring requirements

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 2.15,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.

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 measurable 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*

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 2.15: ASR Expansion Acceptance Criteria and Condition Monitoring Frequencies adapted from Letter SBK-16071, Table 5 (Exh. INT010))

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 modeling 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)**, (**SGH-FEA-4-conf**) 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 2.1.

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 2.18, 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 2.1.

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 (**SBK-PROP00013490**) page 3-2 (**Exh. INT021) (Proprietary):**

   a portion of *the coarse aggregate used for the shear and reinforcement anchorage test specimens was transported by trucks to the laboratory in Texas from a quarry in Maine that is near to the quarry where aggregate was obtained for original construction at Seabrook Station.*

   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

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

    Again, wrong. As previously illustrated, Figure 2.13 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.

    Concerns about steel subsequently clarified

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 % .*

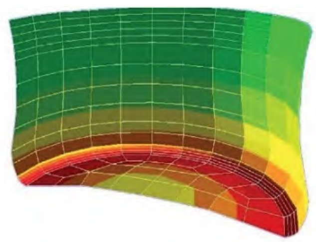
    This is completely arbitrary. There is no scientific basis for such range of values. Expansions vary depending on the source of aggregates, the alkalinity of the cement, the relative humidity, the temperature, and the state of stress. This is a very dangerous and simplistic assumption with no basis.

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

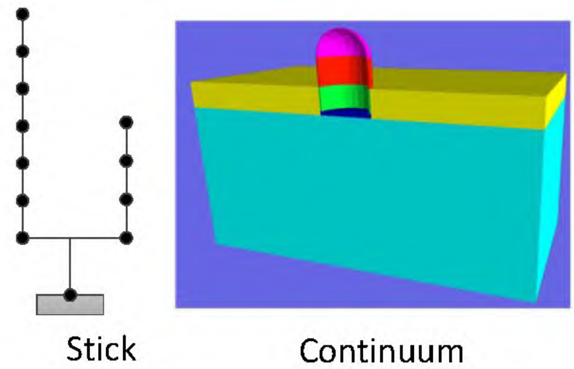
    Again, the stick model is a model of the past when computers did not have sufficient capability to handle the time history analysis of a 3D model, Figure 2.16(b). The stick model is overly simplistic, as discussed in the quote below:

    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. (**ashar2001code**).

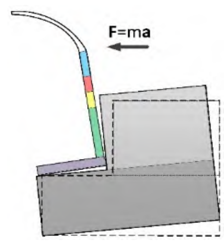
    Furthermore, stick model cannot capture the seismic contact between the wall and the adjacent soil unless joint elements are inserted.



(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 2.16: Concerns about finite element analyses

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 2.3(a).

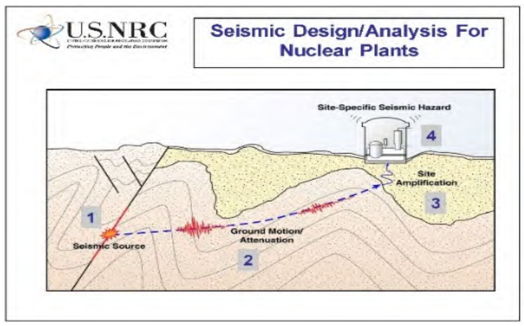
This also implies that cracks will only occur along the direction of the seismic excitation. This is wrong, and shows a poor understanding of structural response of a cylinder subjected to shear force.

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

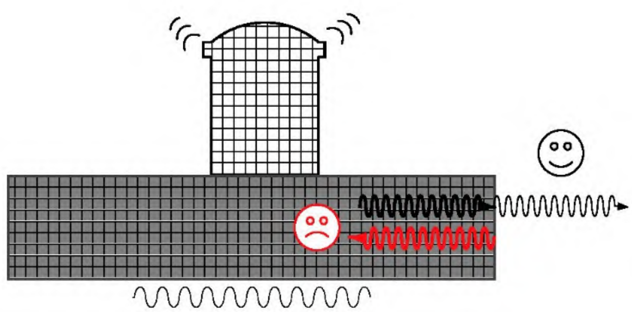
    Because of the linear elastic analysis, the strains are grossly underestimated, best illustrated by Figure 2.1.

19. Impact of soil through proper deconvolution or soil structure interaction is not accounted for as required by **american2016seismic**,

I was subsequently convinced that there was no need to model soil-structure interaction at this location



(a) Seismic Attenuation/Amplification J. (**pires**)



(b) Need for soil-structure interaction in seismic analysis

Figure 2.17: Soil Structures Interaction

C.3.4.2 Why should a Probabilistic Based Analysis be performed for the structural evaluation?

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) (**SGH-Factors**) **(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.

  (**american2016seismic**)

Clearly, and unequivocally, the presence of ASR is a "new information" and it challenges the design basis. But what is probabilistic risk assessment? **beckjord1993probabilistic** 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 (**kemeny1979report**). Finally, after Fukushima, plants were required to reevaluate the potential impact of external events on their structures(**epri2013**) as in some cases the seismic stressor may have been underestimated (**hardy2015us**).

In recent years, there has been a gradual shift toward probabilistic risk evaluations. **kennedy1984seismic** 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, **thomas2006test** is one of the most cited ASR references according to Google Scholar[1]. This test is described below in Figure 2.18.

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.

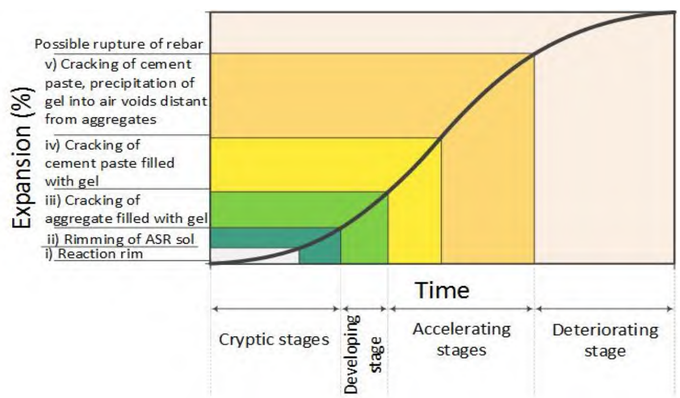
It is also essential to take steps to get good data. This is also a complex process requiring proper testing. First and foremost among them is the use of accelerated expansion tests. This is a well-established test procedure not only for mortar bars but also for concrete. Because AAR is a thermodynamically 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 2.18(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 2.18(b). The test will stop when the expansion has "plateaued" and will provide the maximum expansion likely to occur, and as

[1] This reference was accidentally entered as Fournier, et al. 2006 in the original document.

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

(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 2.18: 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, (**tcherner-09**), (**ikata-3**). 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 $\sim$ $ 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.

Table 2.1 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.

Table 2.1: 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.

Chapter 3

Rebuttal Testimony; August, 2019

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 (**eprilessons**). 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[1] (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, (**ch-aar-swelling**) 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.

[1] Nearly identical to (**chenier2012approach**)

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) (**ise92**);
- 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) (**fhwa10**) ("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.

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, (**epri2018**)), 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 21 st 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 .

Another major concern, is that the 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 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. former DOE/NRC employees

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.

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).

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), (**ulm00**) 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.

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 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?

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 3.1. 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.

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

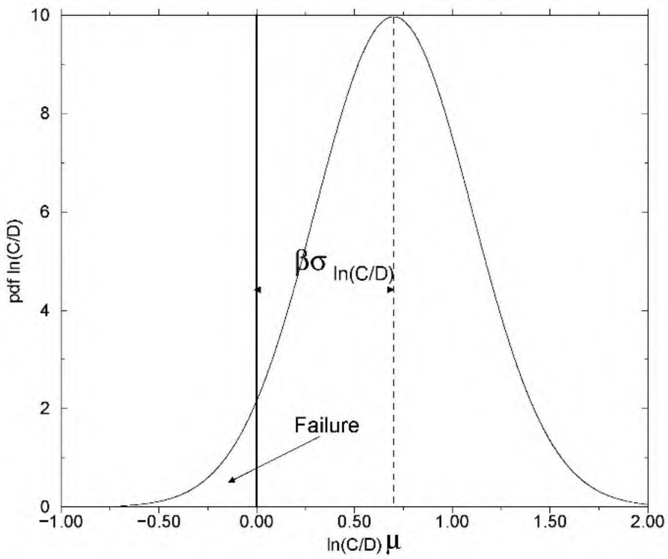


Figure 3.1: Normal distribution of capacity over demand in the definition of the reliability index

with much greater precision. In sum, conservatisms can never overcome fundamental inadequacies in data or analyses.

Relationship between LSTP, SRP and SEM

> D.2.1 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?

Yes. Figure 3.2 below illustrates the tight integration of the LSTP, SEM, and SMP.

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. Hence, the LSTP had a direct impact on the SMP and SEM.

2. Structural component characteristics that affected the subsequent SE:
   (a) Shear capacity

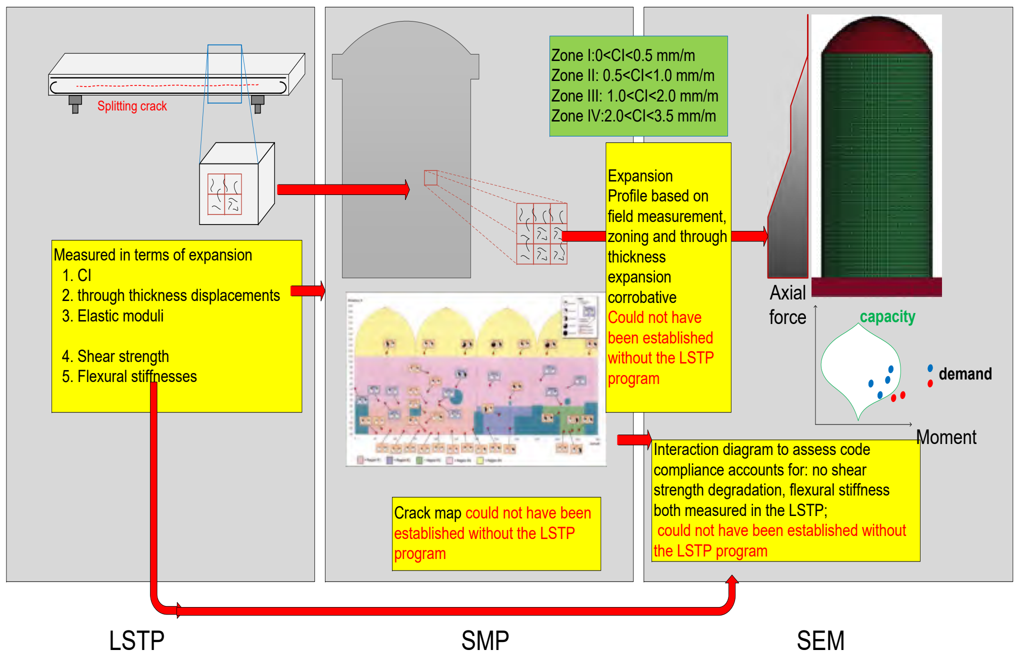


Figure 3.2: Tight integration of LSTP, SM and SEM ([[Portion Redacted]]

(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. Without this observation, the SEM would not have been possible.

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

Representativeness of LSTP

> D.3.1 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.

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 different 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 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.

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 3.3, a depiction of the LSTP derived from **bentz2005**.

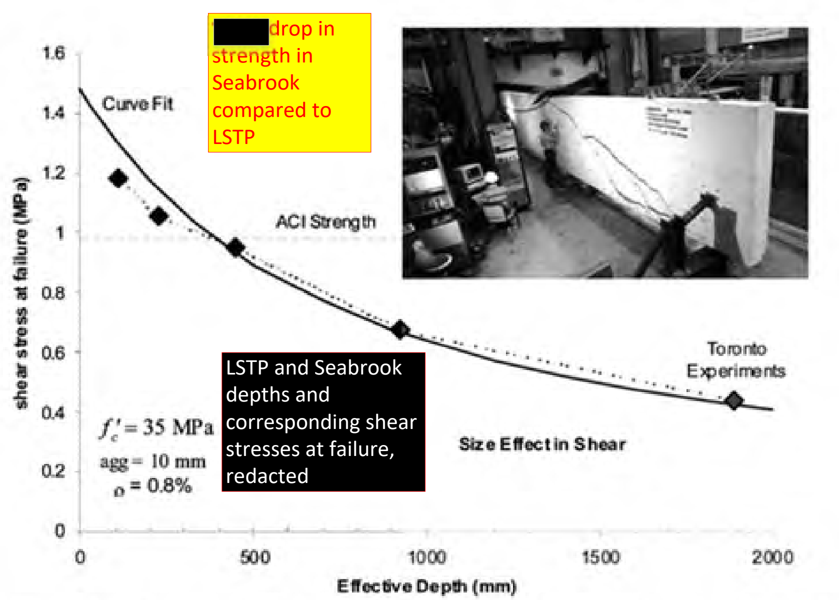


Fig. 1—Example of size effect in shear for reinforced concrete members without stirrups.[1]

Figure 3.3: LSTP is a model test and not a prototype test, there will be a [redacted] reduction in shear strength in Seabrook. Adapted from**bentz2005**.

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.

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 $2 * \sqrt{}$ f'c versus allowable total shear stress of $10 * \sqrt{}$ 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 <u>relative loss</u> 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:

> 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).*

<u>Do you agree?</u>

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 in Figure 3.4. As demonstrated in Figure 3.4, chemical

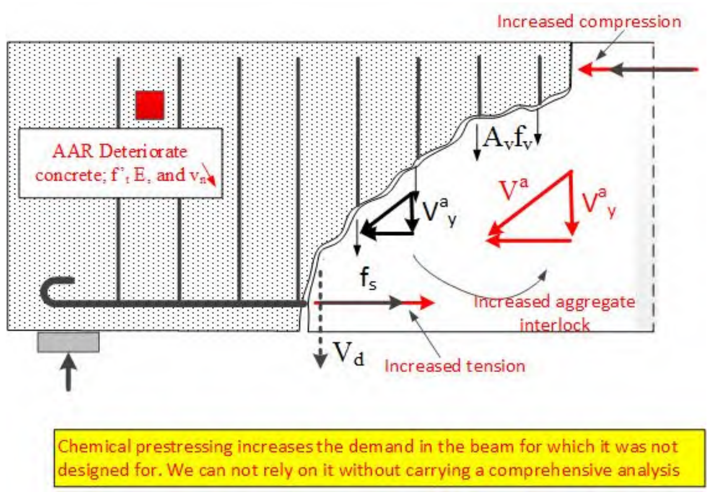


Figure 3.4: Consequences of chemical prestressing

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 (**miyagawa2006fracture**). 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 3.5 below.

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 **inoue2008** 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

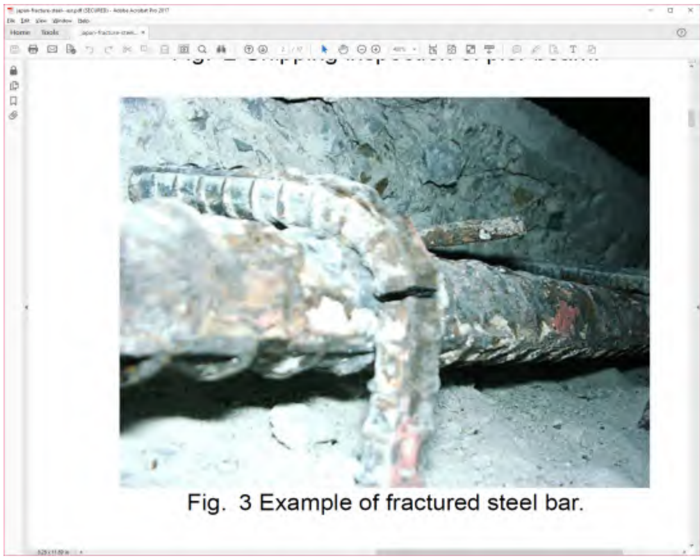


Figure 3.5: Picture of fractured reinforcement as a result of ASR expansion (**miyagawa2006fracture**)

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.

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% (**stark1987alkali**). 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 3.6. The following table

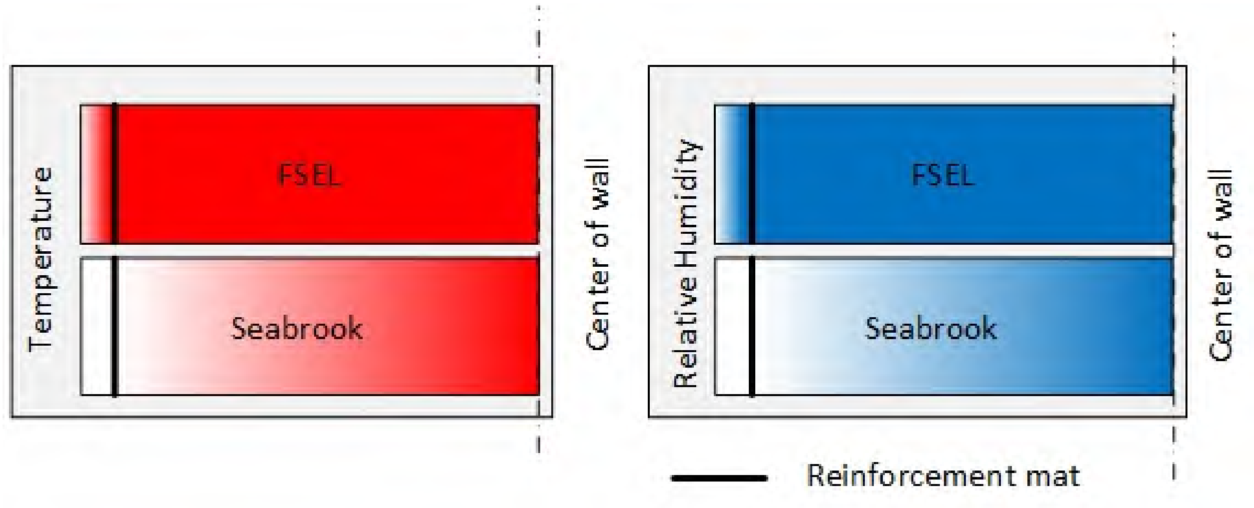


Figure 3.6: Factors affecting progression of ASR

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, Figure 3.7. 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).

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

[redacted]

Figure 3.7: Figure 7 Specimens covered with burlap in the FSEL (Exhibit NER022, MPR-4262 (PROPRIETARY)

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 3.8 (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.

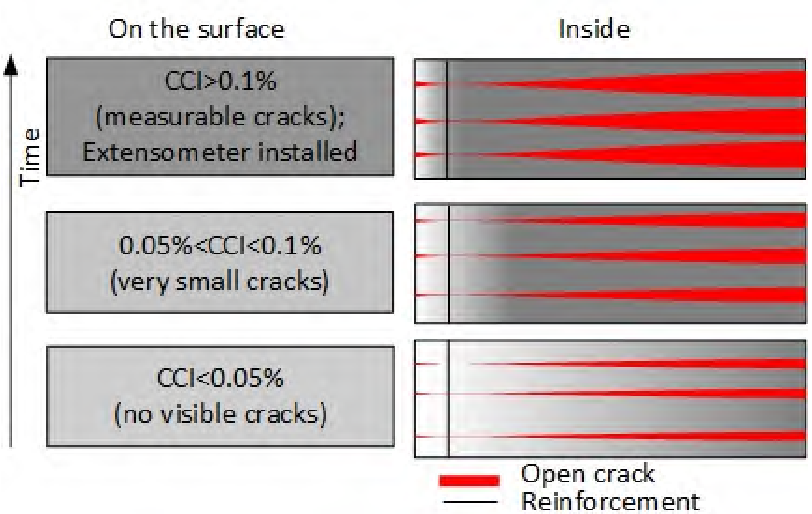


Figure 3.8: Possible scenario of internal cracks formation and unlikelihood of being captured in time by CI readings

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) (**fhwa10**) 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.

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 (**hillerborg1**)), 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 3.9). Here, contrary to metal, it is nearly impossible to observe the tip/front/surface of a FPZ; however, it can be indirectly measured.

Hence, to better understand what has happened a nonlinear 3D fracture mechanics based finite element analysis of the tested beam was conducted (Figure 3.10), 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 3.11. This matches the crack observed in the LSTP, Figure 3.13.

2. At midspan, the computed crack profile is shown in Figure 3.12. 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).

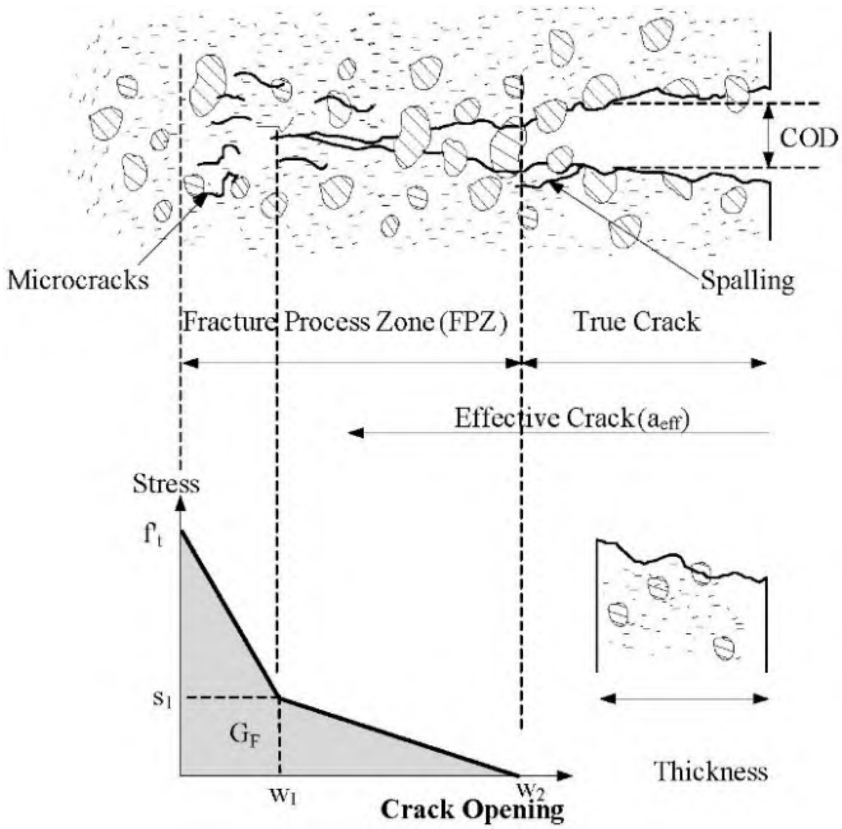
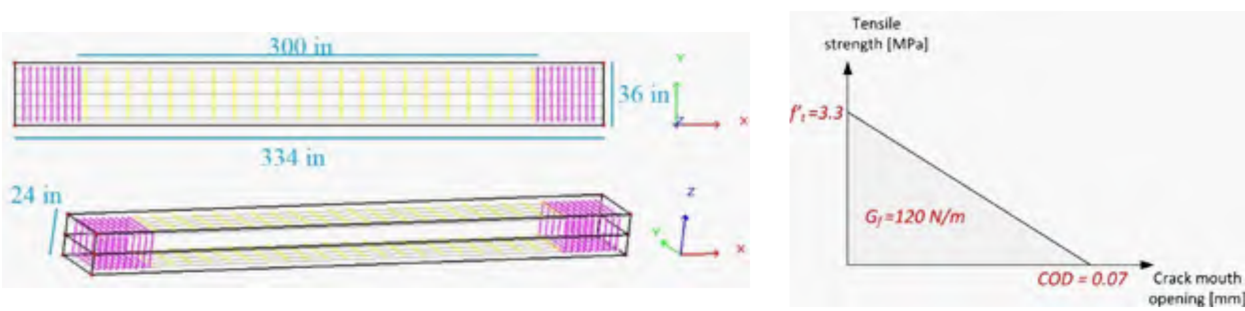


Figure 3.9: Hillerborg's model for concrete, adapted from (**hillerborg1**)



a) b)

Figure 3.10: a) Finite element model analyzed to duplicate pre-test cracks (adapted from (**wald2017expansion**); and b) adopted cohesive crack model

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

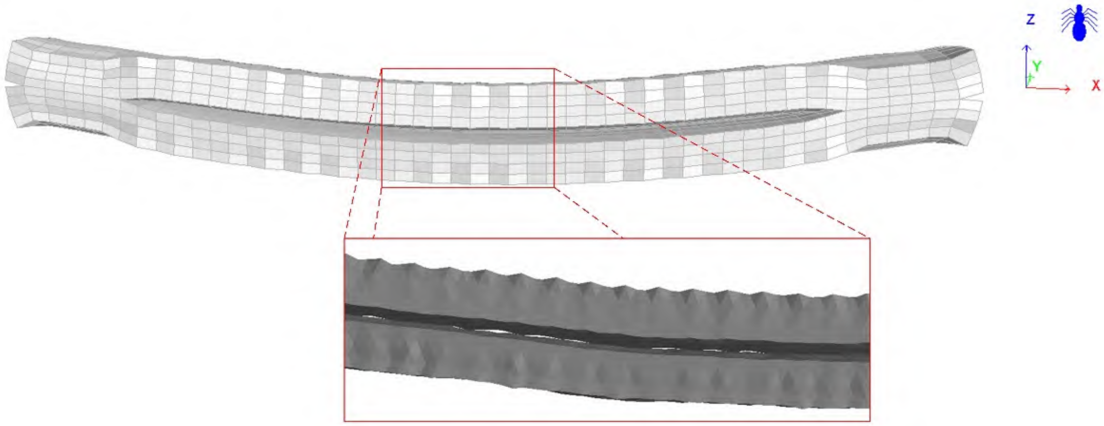


Figure 3.11: Highly amplified deformation showing crack opening in between the stirrups

(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 3.9) 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.

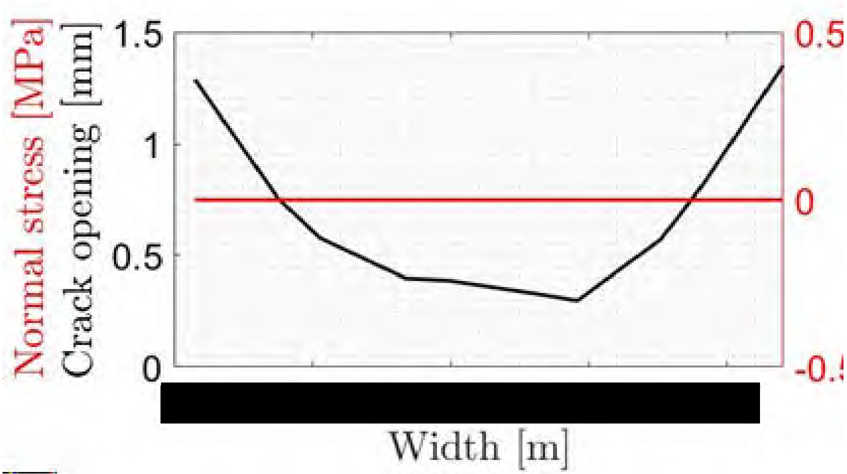


Figure 3.12: Crack opening profile across the beam (along y axis)

The impact of this crack is two-fold:

1. It may have impacted the validity of the shear tests as in Figure 3.13 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 3.13: 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 3.14, 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.

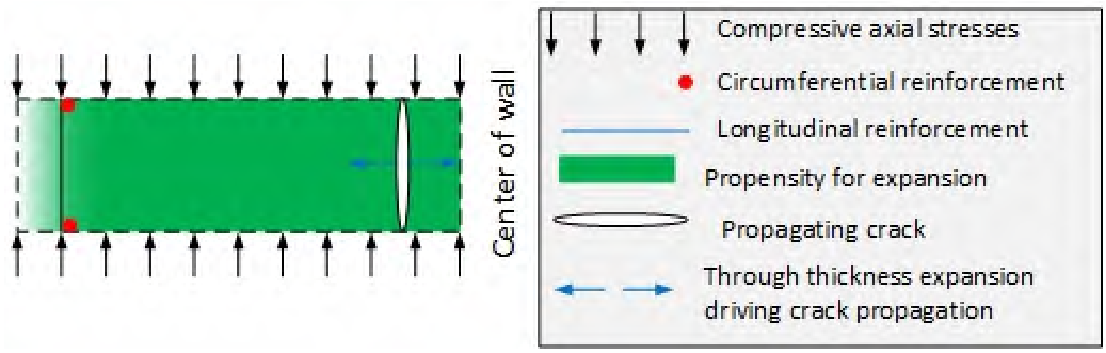


Figure 3.14: Mechanism of internal delamination

6. At some point, those cracks will coalesce (i.e. inter-connect) causing internal delamination, Figure 3.15. 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.)

Should there be any doubt about this, Figure 3.16 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.

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.

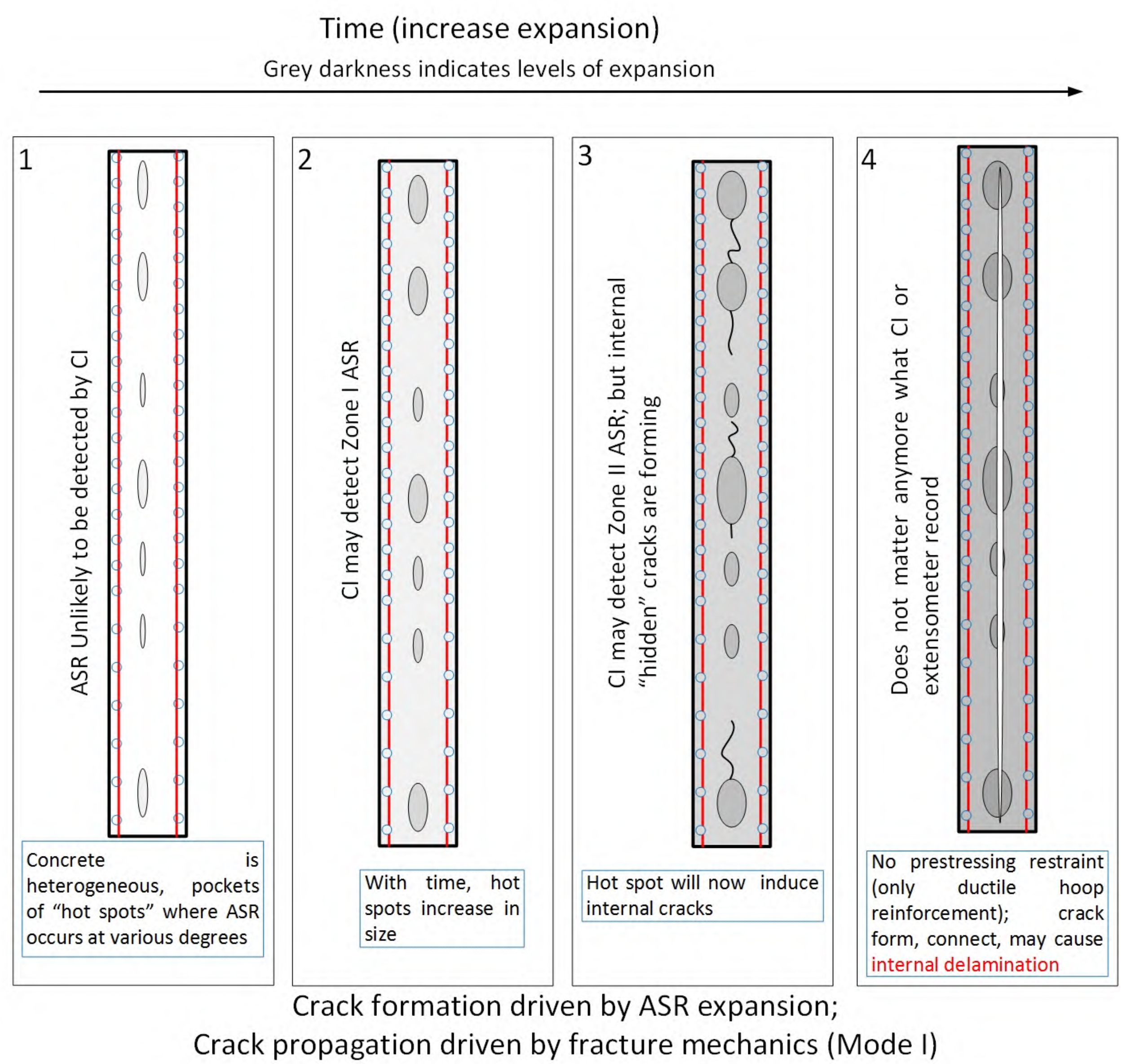


Figure 3.15: Very likely delamination mechanism inside Seabrook

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.

Figure 3.16: Examples of ASR driven delamination in surface reinforced concrete retaining walls in Switzerland (Courtesy Dr. Leemann, EMPA, Switzerland)

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 (**epri2018**), Exhibit NER018.

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 3.17. 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. Indeed **kojima2000** it is reported that "*The reactivity of aggregate*

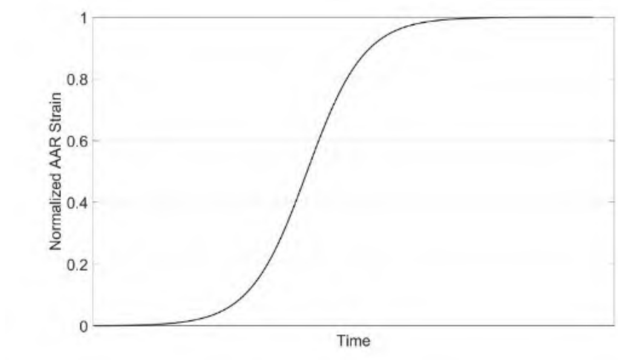


Figure 3.17: Typical expansion curve for ASR

and sudden expansion of the concrete were first discovered in Osaka... ", (emphasis added).

In **dasilva2008** 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 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.

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 3.18. 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.

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.

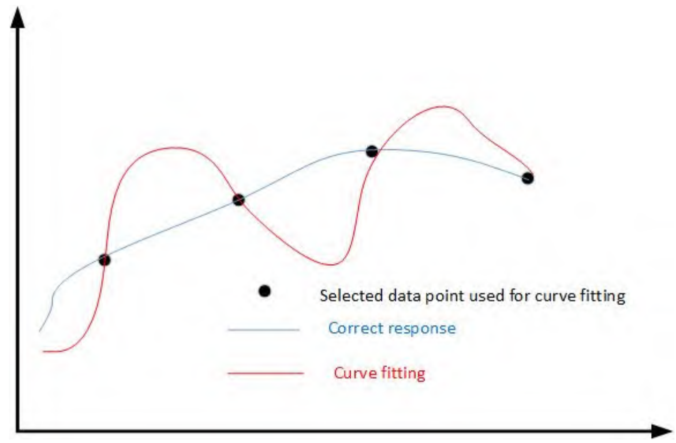


Figure 3.18: Problem with curve fitting

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 (**hansen97b**)

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 **geyskens1998bayesian**. 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 3.19. An analysis of the data shows that the lower and upper bound correspond to [redacted] respectively (as expected).

[redacted]

Figure 3.19: 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 3.20 (Exhibit NRC012, MPR-4153, Rev. 2, "Seabrook Station - Approach for Determining Through Thickness Expansion from Alkali-Silica Reaction," (July 2016)) (PROPRIETARY)

I analyzed the impact of both uncertainties, Figure 3.19 and Figure 3.20 to determine what would be the possible margin of error in NextEra's curve (shown in Figure 3.21). The result is shown in Figure 3.22. The approach is described next:

1. Figure 3.22 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 3.20 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 3.21) 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]

[redacted]

Figure 3.20: 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]

[redacted]

Figure 3.21: NextEra’s “adjusted” correlation curve (note the absence of error bars). [PROPRIETARY INFORMATION]

7. In Figure 3.22 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 underlinevery low confidence 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. 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.

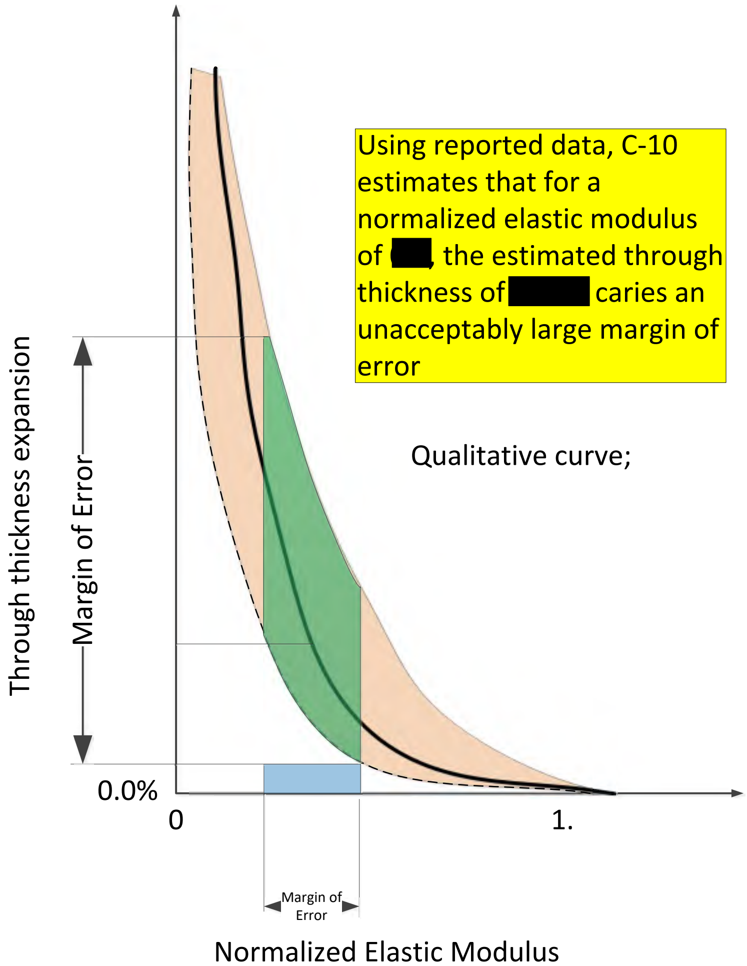


Figure 3.22: Uncertainty in the determination of the Elastic modulus from the compressive strength as proposed by NextEra [~~PROPRIETARY INFORMATION~~]

> 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:
> *underlineThe 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 **eprilessons** 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)[2]. 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.

[2]Nearly identical to (**chenier2012approach**)

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.

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 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 humidity, 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.

Chapter 4

Addendum; September 2019

This Supplemental Rebuttal Testimony relates to the concerns I have raised in Section 3.D.9.1 (page 80) of my Rebuttal Testimony (pages 36-41) regarding weaknesses in the corroboration study described in MPR's testimony at A89, A95, and A176. The corroboration study is used to evaluate "how expansion at the plant aligns with observed expansion of the LSTP specimens." Id. A89. As I testified, MPR's testimony concerns me because it fails to acknowledge the extent to which the corroboration study depends on approximating quantitative values related to ASR. In my testimony, I listed four factors that carry "substantial uncertainties":

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.

I also submitted a graph showing the effects of those uncertainties, Figure 3.22. I concluded that the uncertainties were so large as to be unacceptable.

Since submitting my testimony, I have discovered another fundamental error in NextEra's and MPR's procedure for the corroboration study. This error is due to a key factor that has been overlooked by all parties: the increase in the compressive strength of concrete over time. Failure to acknowledge this phenomenon will result in a corroboration study that underestimates the through-thickness expansion caused by ASR. This error affects my previously submitted Rebuttal Testimony, including Figure 22 on page 40. I am submitting this information now in order to correct my own calculation and to ensure that this very important factor is given consideration by all.

Per NextEra's and MPR's method, the out of plane expansion hinges on their ability to quantify the normalized elastic modulus. This normalized elastic modulus is the ratio of the current (damaged) elastic modulus (measured by ASTM C-469) over the estimated initial (undamaged) elastic modulus at the time of casting (measured by the approximate ACI empirical equation). It should be noted that this estimated elastic modulus is proportional to the (square root of) the compressive strength at casting.

However, MPR does not take into account the very well-known fact that concrete compressive strength increases over time (due to the hydration of the cement), with most of the increase occurring the first few years. Failure to account for the increase in compressive strength will result in an erroneous normalized elastic modulus which would underestimate the through thickness expansion.

Some help can be found in Figure 2.5 of Exhibit NRC-073, a textbook entitled Design of Concrete Structures **nilsonbook**. Figure 2.5 is shown as Figure 4.1. It clearly shows that the compressive strength after five years is 20% higher than the reference (and customary) value measured at 28 days.

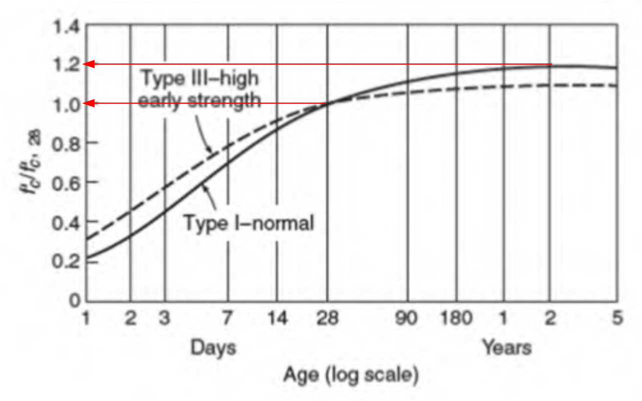


Figure 4.1: Effect of age on compressive strength for moist-cured concrete (adapted from Figure 2.5 of Exhibit NRC-073)

The impact of NextEra's and MPR's oversight is further illustrated in Figure 4.2 below. Taking into consideration the increase in compressive strength during the first two years after casting the Seabrook concrete, the actual amount that compressive strength has been lost due to ASR will be substantially more than the loss estimated by NextEra and MPR using the corroboration study in its current form. In other words, the extent to which ASR has damaged the concrete at Seabrook will be much greater than the extent that will be estimated by MPR's method. Hence, the corroboration study will not yield meaningful results if it fails to account for the fact that the compressive strength of Seabrook concrete was initially much greater than assumed.

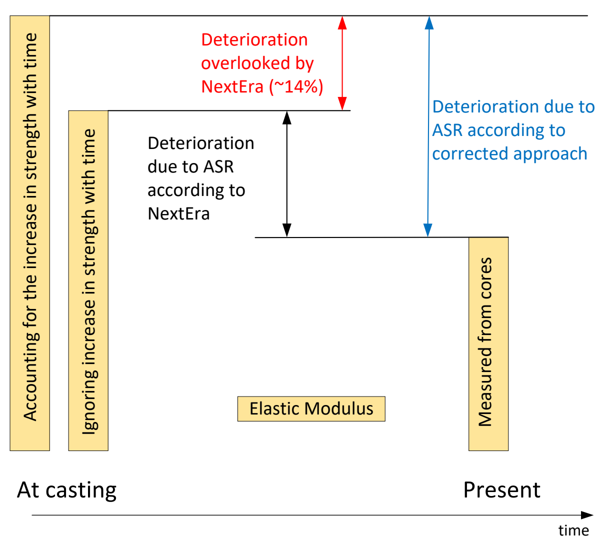


Figure 4.2: Impact of ignoring the concrete strength increase over time.

This new information about the increase in compressive strength also affects my original Figure 22 (page 40 of my Rebuttal Testimony). In Figure 4.3 below, I have modified Figure 22 to account for the increase in the compressive strength. The curve for the normalized elastic modulus is shifted to the left, while maintaining the same margin of error. (Note that I have removed the errors bars to facilitate understanding). Because the through thickness is indeed very sensitive to the normalized elastic modulus (see Figure 21 at page 38 of my Rebuttal Testimony[1]), this will result in an even greater through thickness expansion than predicted by NextEra.

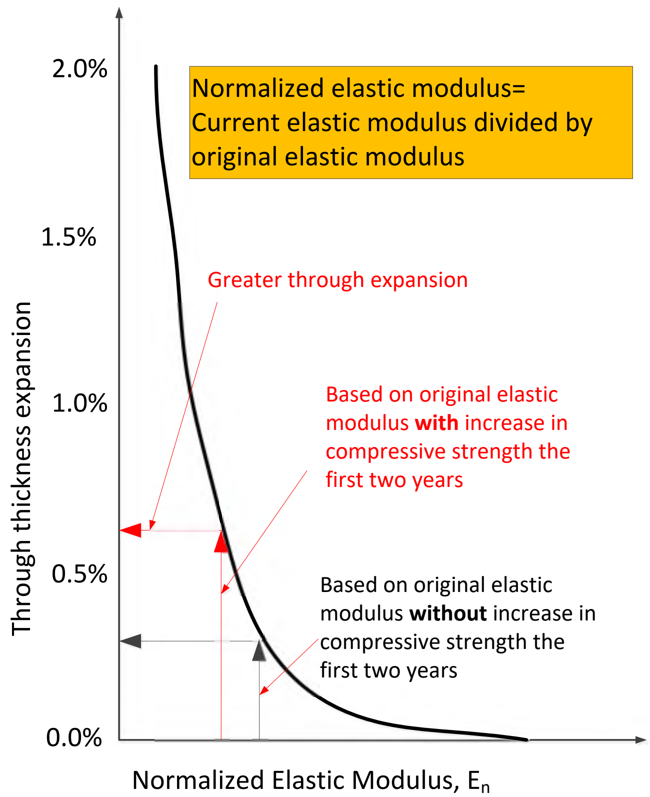


Figure 4.3: Adjusted relationship between through-thickness expansion and the corrected normalized elastic modulus

In summary, this increase in compressive strength of the concrete, and the resulting increase of the initial elastic modulus in the first two years (at a time when ASR would have been infinitesimally small), coupled with the extreme sensitivity of through thickness in terms of normalized elastic modulus.

Failure to account for this phenomenon in the corroboration study constitutes a fundamental flaw. Taken together with the nonconservative nature of the study to which I previously testified, I consider it dangerous to rely on.

[1]This figure has been redacted.

Appendix A

NextEra's License Amendment Request Approval

NextEra's License Amendment Request, was approved by the NRC through Amendment No. 159 to NextEra.

This chapter contains the NRC's (redacted) supporting document.

~~**OFFICIAL USE ONLY – PROPRIETARY INFORMATION**~~

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

March 11, 2019

Mr. Mano Nazar
President and Chief Nuclear Officer
Nuclear Division
NextEra Energy Seabrook, LLC
Mail Stop: EX/JB
700 Universe Blvd.
Juno Beach, FL 33408

SUBJECT: SEABROOK STATION, UNIT NO. 1 – ISSUANCE OF AMENDMENT NO. 159 RE: METHODOLOGY FOR ANALYSIS OF SEISMIC CATEGORY I STRUCTURES WITH CONCRETE AFFECTED BY ALKALI-SILICA REACTION (CAC NO. MF8260; EPID L-2016-LLA-0007)

Dear Mr. Nazar:

The U.S. Nuclear Regulatory Commission (NRC, the Commission) has issued the enclosed Amendment No. 159 to Facility Operating License No. NPF-86 for the Seabrook Station, Unit No. 1. This amendment consists of changes to the license in response to your application dated August 1, 2016, as supplemented by letters dated September 30, 2016; October 3, 2017; October 17, 2017; December 11, 2017; and June 7, 2018.

The amendment revises the Updated Final Safety Analysis Report to adopt a methodology for the analysis of seismic Category I structures with concrete affected by alkali-silica reaction.

The NRC has determined that the related safety evaluation contains proprietary information pursuant to Title 10 of the *Code of Federal Regulations* Section 2.390, "Public inspections, exemptions, requests for withholding." The proprietary information is indicated by text enclosed within double brackets. Accordingly, the NRC staff has also prepared a non-proprietary publicly available version of the safety evaluation, which is provided as Enclosure 2. The proprietary version of the safety evaluation is provided as Enclosure 3.

Enclosure 3 to this letter contains proprietary information. When separated from Enclosure 3, this document is DECONTROLLED.

~~**OFFICIAL USE ONLY – PROPRIETARY INFORMATION**~~

M. Nazar - 2 -

Notice of Issuance will be forwarded to the Office of the *Federal Register* for publication.

Sincerely,

Justin C. Poole, Project Manager
Plant Licensing Branch I
Division of Operating Reactor Licensing
Office of Nuclear Reactor Regulation

Docket No. 50-443

Enclosures:
1. Amendment No. 159 to NPF-86
2. Safety Evaluation (non-proprietary)
3. Safety Evaluation (proprietary)

cc w/o Enclosure 3: Listserv

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

NEXTERA ENERGY SEABROOK, LLC, ET AL.*

DOCKET NO. 50-443

SEABROOK STATION, UNIT NO. 1

AMENDMENT TO FACILITY OPERATING LICENSE

Amendment No. 159
License No. NPF-86

1. The Nuclear Regulatory Commission (the Commission) has found that:

   A. The application for amendment filed by NextEra Energy Seabrook, LLC, et al. (the licensee), dated August 1, 2016, as supplemented by letters dated September 30, 2016; October 3, 2017; October 17, 2017; December 11, 2017; and June 7, 2018, complies with the standards and requirements of the Atomic Energy Act of 1954, as amended (the Act), and the Commission's rules and regulations set forth in 10 CFR Chapter I;

   B. The facility will operate in conformity with the application, the provisions of the Act, and the rules and regulations of the Commission;

   C. There is reasonable assurance: (i) that the activities authorized by this amendment can be conducted without endangering the health and safety of the public, and (ii) that such activities will be conducted in compliance with the Commission's regulations;

   D. The issuance of this amendment will not be inimical to the common defense and security or to the health and safety of the public; and

   E. The issuance of this amendment is in accordance with 10 CFR Part 51 of the Commission's regulations and all applicable requirements have been satisfied.

*NextEra Energy Seabrook, LLC, is authorized to act as agent for the: Hudson Light & Power Department, Massachusetts Municipal Wholesale Electric Company, and Taunton Municipal Lighting Plant and has exclusive responsibility and control over the physical construction, operation and maintenance of the facility.

Enclosure 1

2. Accordingly, by Amendment No. 159, Facility Operating License No. NPF-86 is hereby amended to authorize revision to the Seabrook Station, Unit No. 1, Updated Final Safety Analysis Report (UFSAR), as set forth in the licensee's application dated August 1, 2016, as supplemented by letters dated September 30, 2016; October 3, 2017; October 17, 2017; December 11, 2017; and June 7, 2018, and evaluated in the NRC staff's evaluation enclosed with this amendment.

3. Accordingly, the license is also amended by changes to the paragraph 2.J of Facility Operating License No. NPF-86 and is hereby amended to read as follows:

   J. <u>Additional Conditions</u>

   The Additional Conditions contained in Appendix C, as revised through Amendment No. 159, are hereby incorporated into this license. NextEra Energy Seabrook, LLC, shall operate the facility in accordance with the Additional Conditions.

4. This license amendment is effective as of its date of issuance and shall be implemented within 90 days of issuance. The UFSAR changes shall be implemented in the next periodic update to the UFSAR in accordance with 10 CFR 50.71(e).

FOR THE NUCLEAR REGULATORY COMMISSION

James G. Danna, Chief
Plant Licensing Branch I
Division of Operating Reactor Licensing
Office of Nuclear Reactor Regulation

Attachment:
Changes to the Facility Operating License and Appendix C, Additional Conditions

Date of Issuance: March 11, 2019

ATTACHMENT TO LICENSE AMENDMENT NO. 159

SEABROOK STATION, UNIT NO. 1

FACILITY OPERATING LICENSE NO. NPF-86

DOCKET NO. 50-443

Replace the following page of Facility Operating License No. NPF-86 with the attached revised page. The revised page is identified by amendment number and contains a marginal line indicating the area of change.

Remove	Insert
7	7

Replace the following page of the Appendix C, Additional Conditions, with the attached revised page. The revised page is identified by amendment number and contains a marginal line indicating the area of change.

Remove	Insert
2	2

J. Additional Conditions

The Additional Conditions contained in Appendix C, as revised through Amendment No. 159, are hereby incorporated into this license. NextEra Energy Seabrook, LLC, shall operate the facility in accordance with the Additional Conditions.

K. Inadvertent Actuation of the Emergency Core Cooling System (ECCS)

Prior to startup from refueling outage 11, FPL Energy Seabrook* commits to either upgrade the controls for the pressurizer power operated relief valves (PORV) to safety-grade status and confirm the safety-grade status and water-qualified capability of the PORVs, PORV block valves and associated piping or to provide a reanalysis of the inadvertent safety injection event, using NRC approved methodologies, that concludes that the pressurizer does not become water solid within the minimum allowable time for operators to terminate the event. NextEra Energy Seabrook, LLC submitted an analysis of the inadvertent safety injection event in a letter dated November 7, 2005. In a letter dated June 9, 2006, the NRC concluded the analysis met the requirements of License Condition 2.K.

3. This license is effective as of the date of issuance and shall expire at midnight on March 15, 2030.

FOR THE NUCLEAR REGULATORY COMMISSION

(Original signed by:
Thomas E. Murley)

Thomas E. Murley, Director
Office of Nuclear Reactor Regulation

Attachments/Appendices:
1. Appendix A - Technical Specifications (NUREG-1386)
2. Appendix B - Environmental Protection Plan
3. Appendix C - Additional Conditions

Date of Issuance: March 15, 1990

* On April 16, 2009, the name "FPL Energy Seabrook, LLC" was changed to "NextEra Energy Seabrook, LLC".

AMENDMENT NO. ~~86, 94, 101, 105, 112, 116, 119, 122, 145,~~ 159

Amendment Number	Additional Condition	Implementation Date
	(continued) (b) The first performance of the periodic assessment of CRE habitability, Specification 6.7.6.I.c. (ii), shall be within 3 years, plus the 9-month allowance of SR 4.0.2, as measured from August 2003, the date of the most recent successful tracer gas test, as stated in the December 9, 2003 letter response to Generic Letter 2003-01, or within the next 9 months if the time period since the most recent successful tracer gas test is greater than 3 years. (c) The first performance of the periodic measurement of CRE pressure, Specification 6.7.6.I.d, shall be within 18 months, plus the 138 days allowed by SR 4.0.2, as measured from August 2003, the date of the most recent successful pressure measurement test, or within 138 days if not performed previously.	
159	The licensee will perform the following actions to confirm the continued applicability of the MPR/FSEL large-scale testing program conclusions to Seabrook structures (i.e., that future expansion behavior of ASR-affected concrete structures at Seabrook aligns with observations from the MPR/FSEL large-scale testing program and that the associated expansion limits remain applicable). The licensee shall notify the NRC each time an assessment or corroboration action is completed. (a) Conduct assessments of expansion behavior using the approach provided in Appendix B of Report MPR-4273, Revision 1 (Seabrook FP#101050), to confirm that future expansion behavior of ASR-affected structures at Seabrook Station is comparable to what was observed in the MPR/FSEL large-scale testing program and to check margin for future expansion. Seabrook completed the first expansion assessment in March 2018; and will complete subsequent expansion assessments every ten years thereafter. (b) Corroborate the concrete modulus-expansion correlation used to calculate pre-instrument through-thickness expansion, as discussed in Report MPR-4153, Revision 3 (Seabrook FP#100918). The corroboration will cover at least 20 percent of extensometer locations on ASR-affected structures and will use the approach provided in Appendix C of Report MPR-4273, Revision 1 (Seabrook FP#101050). Seabrook will complete the initial study no later than 2025 and a follow-up study 10 years thereafter.	This amendment shall be implemented within 90 days of March 11, 2019

2 Amendment No. ~~119,~~ 159

ENCLOSURE 2

NON-PROPRIETARY SAFETY EVALUATION

RELATED TO AMENDMENT NO. 159 TO FACILITY OPERATING LICENSE NO. NPF-86

NEXTERA ENERGY SEABROOK, LLC

SEABROOK STATION, UNIT NO. 1

DOCKET NO. 50-443

Proprietary information pursuant to Section 2.390 of Title 10 of
the *Code of Federal Regulations* has been redacted from this document.
Redacted information is identified by blank space enclosed within [[double brackets]].

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

SAFETY EVALUATION BY THE OFFICE OF NUCLEAR REACTOR REGULATION

RELATED TO AMENDMENT NO. 159 TO FACILITY OPERATING LICENSE NO. NPF-86

NEXTERA ENERGY SEABROOK, LLC

SEABROOK STATION, UNIT NO. 1

DOCKET NO. 50-443

1.0 INTRODUCTION

By application dated August 1, 2016 (Reference 1), as supplemented by letters dated September 30, 2016 (Reference 2); October 3, 2017 (Reference 3); October 17, 2017 (Reference 36); December 11, 2017 (Reference 4); and June 7, 2018 (Reference 5), NextEra Energy Seabrook, LLC (NextEra or the licensee) submitted License Amendment Request (LAR) No. 16-03, requesting changes to the Updated Final Safety Analysis Report (UFSAR) for Seabrook Station, Unit No. 1 (Seabrook).

The LAR proposed to include methods for analyzing seismic Category I structures with concrete affected by alkali-silica reaction (ASR). The LAR states that the design codes for the affected structures do not account for the impacts of ASR; therefore, the proposed methodology changes, and supporting technical bases are necessary to reconcile the design basis of the containment building and other seismic Category I concrete structures that are affected by ASR.

Portions of the letters dated August 1, 2016; September 30, 2016; and October 3, 2017, contain sensitive unclassified non-safeguards information (i.e., proprietary information) and, accordingly, those portions have been withheld from public disclosure.

The supplemental letters dated October 3, 2017; October 17, 2017; December 11, 2017; and June 7, 2018, provided additional information that clarified the application, did not expand the scope of the application as originally noticed, and did not change the U.S. Nuclear Regulatory Commission (NRC) staff's original proposed no significant hazards consideration determination as published in the *Federal Register* on February 7, 2017 (82 FR 9604). Please note that the staff's safety evaluation (SE) contains licensee proprietary information and is thus marked accordingly with double square brackets (**[[ ]]**).

2.0 REGULATORY EVALUATION

Title 10 of the *Code of Federal Regulations* (10 CFR) paragraph 50.59(c)(2)(viii) requires a licensee to obtain a license amendment pursuant to 10 CFR 50.90, "Application for amendment of license, construction permit, or early site permit," prior to implementing a proposed change if the change would "[r]esult in a departure from a method of evaluation described in the FSAR

[Final Safety Analysis Report] (as updated) used in establishing the design bases or in the safety analyses."

In accordance with 10 CFR 50.59, "Changes, tests, and experiments," and 10 CFR 50.90, the licensee requested to amend its license to revise the Seabrook UFSAR to include methods for analyzing and evaluating seismic Category I structures with concrete affected by ASR. These seismic Category I structures were designed and constructed to the requirements of American Concrete Institute (ACI) 318-71, with the exception of the containment building, which was designed and constructed in accordance with the requirements of Section III, Division 2, of the 1975 Edition of the American Society of Mechanical Engineers Boiler and Pressure Vessel Code (ASME Code). The provisions of the design/construction codes of record (ACI 318-71 and the ASME Code), written in the context of new design and construction, did not consider ASR-degraded concrete, and do not include provisions for the analysis and design evaluation of structures affected by ASR. Therefore, the licensee is proposing changes to the method of evaluation, with supporting technical bases, to: (1) reconcile the licensing design basis of the structures impacted by ASR, and (2) demonstrate that the structures continue to meet the acceptance criteria in the respective code of record, as justified to be applicable and supplemented or modified in the LAR. The NRC staff reviewed the proposed changes to verify that the design basis, as modified, continues to meet the requirements of the respective codes, as justified and supplemented or modified by the LAR, for applicability, adequacy, and sufficiency. The safety analysis of Seabrook seismic Category I structures is described in UFSAR Chapter 3, Section 3.7, "Seismic Design," and Section 3.8, "Design of Category I Structures" (Reference 6).

Enclosure 1 (proprietary; Enclosure 7, non-proprietary version), Section 2.2, "Proposed Changes to UFSAR," of the letter dated August 1, 2016 (Reference 1), notes that the UFSAR is revised to allow seismic analysis results to be combined using the "100-40-40" procedure from NRC Regulatory Guide (RG) 1.92, Revision 3, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," dated October 2012 (Reference 7), for analyses of ASR loads. During the course of its review, the licensee determined that the "100-40-40" procedure, as discussed in RG 1.92, was not necessary, and the licensee removed the use of RG 1.92, Revision 3, from the application.

As indicated in Enclosure 1, Section 4.1, "Applicable Regulatory Requirements/Criteria," of the letter dated August 1, 2016, and described in UFSAR Section 3.1, "Conformance to NRC General Design Criteria," Seabrook is committed to 10 CFR Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants." The general design criteria (GDC) that are applicable to the UFSAR changes proposed in the LAR are GDC 1, 2, 4, 16, and 50. Seabrook must continue to meet these criteria with the implementation of the proposed changes. Of these, GDC 1, 2, and 4 apply to all Seabrook seismic Category I structures, including containment; GDC 16 and 50 apply only to the containment. Below is a summary of each of the GDC applicable to the proposed changes.

<u>Criterion 1 – Quality standards and records</u>

Criterion 1 states, in part, that:

> Structures, systems, and components important to safety shall be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. Where generally recognized codes and standards are used, they shall be identified and evaluated to

determine their applicability, adequacy, and sufficiency and shall be supplemented or modified as necessary to assure a quality product in keeping the with the required safety function. A quality assurance program shall be established and implemented in order to provide adequate assurance that these structures, systems, and components will satisfactorily perform their safety functions.

Criterion 2 – Design bases for protection against natural phenomena

Criterion 2 states that:

> Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The design bases for these structures, systems, and components shall reflect: (1) appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, (2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena and (3) the importance of the safety functions to be performed.

Criterion 4 – Environmental and dynamic missile design bases

Criterion 4 states, in part, that:

> Structures, systems, and components important to safety shall be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including loss-of-coolant accidents. These structures, systems, and components shall be appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit.

Criterion 16 – Containment design

Criterion 16 states that:

> Reactor containment and associated systems shall be provided to establish an essentially leak-tight barrier against the uncontrolled release of radioactivity to the environment and to assure that the containment design conditions important to safety are not exceeded for as long as postulated accident conditions require.

Criterion 50 – Containment design basis

Criterion 50 states, in part, that:

> The reactor containment structure … shall be designed so that the containment structure and its internal compartments can accommodate, without exceeding the

design leakage rate and with sufficient margin, the calculated pressure and temperature conditions resulting from any loss-of-coolant accident. This margin shall reflect consideration of ... the conservatism of the calculation model and input parameters.

Appendix B to 10 CFR Part 50, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants"

In addition, activities related to the changes proposed in the LAR are subject to the applicable quality assurance requirements of 10 CFR Part 50, Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants." These include procurement control measures on purchased materials, equipment, services, and design control measures. Section III, "Design Control," of Appendix B to 10 CFR Part 50, requires that the design control measures shall be established to assure that applicable regulatory requirements and the design basis, as defined in 10 CFR 50.2, "Definitions," and as specified in the LAR for applicable structures, are correctly translated into specifications, drawings, procedures, and instructions. These measures shall include provisions to assure that appropriate quality standards are specified and included in design documents and that deviations from such standards are controlled. Design changes, including field changes, shall be subject to design control measures commensurate with those applied to the original design.

The proposed design-bases change, as a result of this LAR, is the addition of ASR and its effects as a design-basis load. Of the applicable GDC, GDC 1 and 2 are the criteria that are directly impacted by the proposed changes, because they address the design-bases loads and load combinations and the use of codes and standards to demonstrate that intended safety functions will be accomplished under those loads and load combinations. The design loads and/or functions defined by GDC 4, 16, and 50 remain unchanged as a result of ASR and are included in the load combinations defined in the current licensing basis. Therefore, if GDC 1 and 2 are met with the proposed changes, the other GDC will also be met.

The NRC staff reviewed the proposed method of analysis in accordance with the listed GDC and the relevant acceptance criteria in Sections 3.8.1, 3.8.3, 3.8.4, and 3.8.5 of NUREG-0800, "Standard Review Plan [SRP] for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR [Light-Water Reactor] Edition." The NRC staff notes that there is no precedent for evaluating the effects of ASR on structural performance of the affected structures and, therefore, this LAR involves unique, plant-specific, and first-of-a-kind review considerations regarding establishing the ASR load, the technical bases for evaluation of ASR-affected concrete structures, and related structural monitoring that are not covered by the construction codes of record or the guidance in NUREG-0800. As required by GDC 1, the staff reviewed the plant-specific technical bases and code supplements or modifications provided in the LAR for applicability, adequacy, sufficiency, and limitations for the use of the codes of record (ACI 318-71 and ASME Code, Section III, Division 2) to evaluate Seabrook concrete structures affected by ASR.

3.0 TECHNICAL EVALUATION

3.1 Background

3.1.1 Description of ASR

The ASR occurs in concrete, in the presence of moisture, when reactive silica that may be present in the aggregate reacts with hydroxyl ions (OH-) and alkali ions (Na+, K+) in the pore solution. The reaction produces an alkali-silicate gel that expands as it absorbs moisture, resulting in micro-cracking in the concrete. The amount of gel depends on the amount and type of silica and alkali hydroxide concentration, and the amount of cracking is dependent on the geometry and reinforcement detailing of each structural member. Typical cracking caused by ASR is described as "pattern" or "map" cracking and is usually accompanied by dark staining adjacent to the cracks. Although visual indications can suggest the presence of ASR, the reaction can only be confirmed via petrographic analysis of cores from affected concrete.

In order for the reaction to occur, all three of the following conditions must be present: reactive forms of silica in the concrete aggregate; high-alkali cement pore solution; and adequate moisture (typically approximately 80 percent or higher relative humidity). If one of these three conditions is absent, the reaction will not proceed. If the reaction occurs, the resulting cracking degrades the mechanical material properties (compressive strength, elastic modulus, tensile strength) of the affected concrete and may necessitate a structural evaluation. In general, ASR deterioration is slow, and the risk of catastrophic failure is low. However, the ASR-induced expansion can cause serviceability problems and may potentially aggravate other concrete deterioration mechanisms, such as reinforcement corrosion, and could impact structural performance. The progression of ASR over time can also result in macro manifestations such as discrete cracking and building deformation.

3.1.2 ASR at Seabrook

As noted in Enclosure 1, Section 2.1, "Background on ASR at Seabrook Station," of the letter dated August 1, 2016 (Reference 1), the licensee initially identified visual indications (i.e., pattern cracking) typical of ASR in the B Electrical Tunnel in 2009, and subsequently in other seismic Category I structures. To verify the presence of ASR, petrographic analysis was completed on concrete cores removed from several affected plant structures, which confirmed ASR. The licensee's root cause investigation concluded that the original concrete mix contained a coarse aggregate that was a slow reactive aggregate (appropriate testing at the time was unable to detect this type of reactivity). This, in combination with groundwater intrusion issues for below-grade structures or other moisture sources during the plant life, resulted in the observed ASR in several structures. The expansion and cracking of concrete from ASR can potentially impact both structural capacity (i.e., load carrying capacity for critical limit states) and the demand (i.e., load due to internal and/or external restraint) on a structure.

Enclosure 1, Section 2.1.1, "Evaluation of ASR at Seabrook," of the letter dated August 1, 2016, notes that in 2012, an interim assessment (Reference 8) was completed, which evaluated the structural adequacy of buildings impacted by ASR. The assessment determined that the structures at Seabrook remain suitable for service for an interim period, given the extent and rate of ASR identified. However, the assessment noted that additional work needed to be done to verify that the structures satisfy the ACI 318-71 (Seabrook's design code) requirements. To address this, NextEra and its consultant (MPR Associates) conducted a large-scale testing program (LSTP) at the Ferguson Structural Engineering Laboratory (FSEL) of the University of

Texas at Austin (the test program is hereafter referred to as the MPR/FSEL LSTP), which was completed in 2016. Using the results from the test program and literature, NextEra developed a method for evaluating and monitoring ASR-affected concrete structures.

Section 2.1.1 also notes that in 2014, a torn seismic gap seal was identified between the containment enclosure building (CEB) and the containment building. The licensee determined that this degradation was caused by relative building movement due to radial deformation of the CEB from ASR expansion of concrete in the CEB and the concrete backfill that abuts the CEB. After discovery of the deformation, NextEra assessed ASR-affected structures, and its prompt operability determinations concluded that the structures and concrete anchors are operable but degraded and nonconforming, and that structures, systems, and components (SSCs) housed within the structures are operable. The LAR proposes an analysis methodology to incorporate the material effects (in the structural context) and loads of ASR into the Seabrook design basis to demonstrate that structures with ASR continue to meet the requirements of the design code, as supplemented by the LAR.

The LAR proposes to revise the UFSAR to include methods for the analysis and design evaluation of seismic Category I structures with concrete affected by ASR. These methods are largely based on the results of the MPR/FSEL LSTP, in combination with field measurements and observations of ASR effects on affected structures. The NRC staff's review assesses the testing (as a technical basis for applicability of code provisions and limitations) and the resultant methodology. Accordingly, this SE has been divided into four main topics addressing: (1) the MPR/FSEL LSTP development, conclusions, and application to Seabrook structures; (2) the proposed method of evaluation for ASR-affected structures; (3) the proposed method of monitoring ASR progression; and (4) the proposed UFSAR changes. All four of these topics are discussed in detail in the following technical evaluation sections.

3.2 MPR/FSEL LSTP and Results (Reference 1, Enclosure 5 (Proprietary; Enclosure 2, Non-Proprietary Version) and Enclosure 6 (Proprietary; Enclosure 3, Non-Proprietary Version); Reference 2, Enclosure 5 (Proprietary; Enclosure 3, Non-Proprietary Version))

Enclosure 1, Section 2.2, of the letter dated August 1, 2016 (Reference 1), proposes a change to the UFSAR that will allow structural evaluations of ASR-affected concrete structures to use the nondegraded, specified concrete material properties and code equations from the original design analyses when ASR expansion levels remain below the levels identified in UFSAR Section 3.8.4.7, "Testing and In-Service Surveillance Requirements," which are based on the results of the MPR/FSEL LSTP. Enclosure 1, Section 3.2, "Impact of ASR on Seabrook Structures," discusses the MPR/FSEL LSTP conducted to develop the technical bases to support the objectives of NextEra's evaluation of the effects of ASR on Seabrook structures with regard to: (a) load carrying capacity for critical structural limit states and other design considerations to demonstrate that Seabrook structures with ASR meet the strength requirements of ACI 318-71 (the design code of record) and (b) identifying parameters and methods for effective monitoring of ASR. The LAR notes that the need for this Seabrook-specific program was driven by the limitations and gaps in the publicly available test data related to ASR effects on structures. Most research on ASR has focused on the science and kinetics of ASR rather than engineering research on structural implications under load. Although structural testing of ASR-affected test specimens has been performed by other researchers, the application of the results and conclusions of the publicly available literature to a specific structure can be challenged by lack of representativeness in the data (e.g., small-scale specimens, different reinforcement configuration, lack of structural context). The MPR/FSEL LSTP included test specimens that reflected the characteristics of ASR-affected structures at Seabrook, and test data were obtained across a range of ASR levels that exceed and bound the levels currently found in Seabrook structures (i.e., more severe), using a common methodology and identical specimens (ASR-affected and control). The MPR/FSEL LSTP was intended to supplement code requirements and used test methods consistent with the test data that were relied upon in developing the ACI 318 code provisions for shear and reinforcement anchorage. The MPR/FSEL LSTP provided improved data on the limit states that were essential for evaluating seismic Category I structures at Seabrook. The results were used to assess the structural limit states and to inform the assessment of other design considerations.

The details of the MPR/FSEL LSTP development and the test results are described in Report MPR-4273, Revision 0, "Seabrook Station – Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction," dated July 2016 (Seabrook FP#101050, Proprietary), included as Enclosure 6 to the letter dated August 1, 2016 (Enclosure 3 is the non-proprietary version). The implications of the test results to Seabrook structures are discussed in Report MPR-4288, Revision 0, "Seabrook Station: Impact of Alkali-Silica Reaction on the Structural Design Evaluations," dated July 2016 (Seabrook FP#101020, Proprietary), included as Enclosure 5 to the letter dated August 1, 2016 (Enclosure 2 is the non-proprietary version). The MPR/FSEL LSTP also developed a methodology for correlating expansion (through-thickness) measured in the test specimens to Seabrook structures. This methodology is used for determining the through-thickness expansion to date at locations of interest in affected structures at Seabrook prior to installation of extensometers and is described in Report MPR-4153, Revision 2, "Seabrook Station – Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction," July 2016 (Seabrook FP# 100918, Proprietary), included as Enclosure 5 to the letter dated September 30, 2016 (Enclosure 3 is the non-proprietary version). In the following sections, the NRC staff reviews the test program as a whole, including the conclusions and their application to Seabrook structures.

3.2.1 Plant-Specific MPR/FSEL LSTP Development

Report MPR-4273, Revision 0 (Enclosure 6 of the letter dated August 1, 2016 (Reference 1)), provides a summary of the plant-specific MPR/FSEL LSTP, including the purpose, setup, and results of the test program. The MPR/FSEL LSTP consisted of three key test program elements that conducted load tests to failure to evaluate the impact of ASR: (1) on performance of expansion and undercut anchors installed in concrete (Anchor Test Program), (2) on shear capacity of reinforced concrete (Shear Test Program), and (3) on reinforcement anchorage of rebar lap splices and flexural strength and stiffness (Reinforcement Anchorage Test Program). These three key elements were chosen based on an interim structural assessment for Seabrook structures that identified these limit states as areas where gaps existed in available literature or available margins in Seabrook structures were low, and for which it was necessary to develop structural performance data under load to complete followup structural evaluations of Seabrook ASR-affected structures. Additionally, a fourth test program (Instrumentation Test Program) evaluated instruments for measurement of through-thickness expansion on Seabrook structures.

Section 1.3, "Commercial Grade Dedication," of Report MPR-4273, Revision 0, discusses commercial grade dedication of the MPR/FSEL LSTP, which was conducted by FSEL under technical direction and quality assurance oversight from NextEra's contractor, MPR Associates, in accordance with the MPR nuclear quality assurance program guidance. The testing was governed by MPR test specifications and conducted under FSEL's project-specific quality system manual using test procedures approved by MPR. MPR commercially dedicated the testing services performed by FSEL and prepared commercial grade dedication reports for the four test programs of the MPR/FSEL LSTP.

Section 2, "Selection of Approach for Test Programs," of Report MPR-4273, Revision 0, discusses how the test program was developed and notes that a literature review indicated that removed cores from ASR-affected concrete will show a reduction in material properties, but this reduction does not necessarily reflect a decrease in structural capacity of a reinforced concrete structural component or system. The presence of two-dimensional reinforcement mats, like those in typical Seabrook structures, or three-dimensional reinforcement (e.g., lower elevations of containment near the base) provides confinement, restraining expansion, and deleterious cracking. This differentiates the structural performance of ASR-affected reinforced structures from unreinforced structures that are more accurately represented by cores. Load testing full-scale specimens with similar reinforcement to Seabrook structures provides much more representative results than simpler approaches that do not account for confinement.
Section 2.4, "Test Program Considerations," of Report MPR-4273, Revision 0, discusses two methods that were considered for developing the test specimens. One involved harvesting specimens from existing ASR-affected structures, while the other involved fabricating specimens and accelerating ASR development. Table 2-1, "Comparison of Test Specimen Approaches," of Report MPR-4273, Revision 0, lists the advantages and disadvantages of each approach and notes that harvested specimens allow ASR to develop naturally over a slow timescale (more realistic to actual ASR progression), but the harvesting process may damage the specimens, and the test is limited to the ASR levels at the time of harvesting. Fabricated specimens allow control of test variables and testing beyond ASR levels exhibited in the actual structures, as well as a common basis for comparison relative to ACI code provisions; however, the ASR development is much quicker than in the actual structures. The licensee chose to use fabricated specimens so that the impact of ASR could be determined as a function of ASR severity, and ASR levels beyond that currently observed on Seabrook structures could be investigated to account for and bound effects of potential future progression of ASR at Seabrook.

Section 3, "Test Specimen Configuration," of Report MPR-4273, Revision 0, discusses the design of the specimens and notes that they were designed with features that represent the reinforced concrete structures at Seabrook to the maximum extent possible. The specimens were of large size to represent the scale and structural context of structures at Seabrook. The MPR/FSEL LSTP used test methods and experimental designs consistent with those that formed the bases of the licensing basis standards of Seabrook Station (i.e., ACI 318-71 for reinforcement anchorage and shear capacity testing, and response to NRC Inspection and Enforcement (IE) Bulletin No. 79-02, "Pipe Support Base Plate Designs Using Concrete Expansion Anchor Bolts," Revision 2 (Reference 9), for anchor capacity testing). The specimens were designed with reinforcement ratios and configurations similar to the layout at Seabrook. To the extent practical, concrete constituents for the beams were obtained from sources similar to those used during the construction of the plant. **[[**

]]. The concrete mix design for the test specimens was based on specifications used at Seabrook (e.g., compressive strength, coarse aggregate gradation and type, water-to-cement ratio, cement type, aggregate proportions). The reinforcement configuration consisted of a two-dimensional rebar mat in the in-plane or x-y direction to simulate the rebar in the face of the typical walls at Seabrook, and **[[**

]]. Additional details on the test specimens can be found in Table 3-1, "Comparison of Fabricated Test Specimens," of Report MPR-4273, Revision 0.

To accelerate ASR development and enable testing at various ASR levels, reactive fine aggregate was used in the concrete **[[ ]]**. The anchor program consisted of **[[ ]]** large-scale blocks and two existing ASR-affected bridge girders that allowed for a total of **[[ ]]** anchor tests. The shear program consisted of **[[ ]]** specimens with a total of **[[ ]]** tests, and the reinforcement anchorage program included **[[ ]]** specimens and **[[ ]]** tests.

Section 4, "Characterizing ASR Development," of Report MPR-4273, Revision 0, discusses how ASR development was characterized and tracked during the test program. The objective of each test program was to develop a trend for structural capacity (determined by load test to failure) as a function of ASR distress levels. Therefore, accurate characterization of ASR levels developed in the test specimens was essential. Expansion was monitored in two directions on the surface adjacent to the reinforcement (i.e., in-plane or x-y direction) along with the direction normal to the reinforcement (i.e., through-thickness or z direction). In addition, concrete cylinders were fabricated and cores were taken from specimens and tested for compressive strength, elastic modulus, and tensile strength to quantify ASR degradation. Petrographic analysis was also conducted on the cores taken from the test specimens just prior to load testing to assess the general properties of the concrete and to confirm the presence of ASR. The in-plane expansion in the test specimens was determined by measured crack indexing (CI) or combined crack indexing (CCI), which is the average of CI in the two directions when of comparable magnitude) and/or by measurement of distance between embedded pins. The through-thickness expansion in the test specimens was determined based on measurement of distance between embedded pins.

The NRC staff reviewed the information on the development of the plant-specific test program provided in the letter dated August 1, 2016, and Report MPR-4273, Revision 0 (Enclosure 6 of Reference 1). The NRC staff noted that the research program focused on structural limit states where gaps existed and additional performance data in the structural context was necessary, or where Seabrook structures appeared to have less margin as determined by the interim structural assessment. Implications of ASR on each of these limit states is discussed below, including a discussion of the limit states that were not focused on in the research program. The NRC staff noted that the licensee used large-scale specimens with concrete constituents, mix design, and reinforcement details that were similar to a Seabrook reference location and representative of the mechanical behavior of typical Seabrook structures. NRC staff also recognized the necessity to include more highly reactive aggregate and alkali material to accelerate ASR. The size and reinforcement of the specimens provide realistic structural context that minimize many uncertainties that may have been added due to scaling effects and allow the structural performance tests to account for the confinement effects provided by the reinforcement. The NRC staff noted that in the structural context (i.e., a composite reinforced concrete structural system), ASR imparts a prestressing effect (inducing a tensile stress in the reinforcement, which induces an equal compressive stress in the concrete) as a result of confinement or restraint to ASR expansion provided by the reinforcement. Therefore, in order to provide a more realistic and representative assessment, evaluations of ASR-affected reinforced concrete structures must take into account the structural context rather than relying on material testing alone. Developing specimens to mimic Seabrook structures as closely as reasonably achievable provides assurance that the results of the testing will be more representative of Seabrook structures than existing ASR research, and more representative and realistic than material testing of unconfined cores. The number of tests and specimens is also reasonable, considering the size of the specimens and ASR expansion being the primary test variable for each test program. Although smaller-scale specimens may have allowed for more specimens and associated tests, uncertainties would have been introduced due to scale effects. The number of specimens tested allowed the licensee to investigate the impact of ASR on structural performance over a series of expansion levels that bound expansion seen to date at Seabrook and account for effects of potential future expansion.

The NRC staff noted that Figure 4-2 in Report MPR-4273, Revision 0, depicts a large crack on the surface of the specimen midway between the reinforcement mats. This crack was an "edge effect" where expansion was concentrated into a large crack due to a lack of confinement. It was also noted that by sectioning three beam specimens after load testing, the licensee confirmed that the large crack observed on the surfaces between the reinforcement mats was an edge effect that penetrated only a few inches into the specimen and did not compromise the representativeness of the test region. A single large crack due to the edge effect is not expected to occur on Seabrook structures due to confinement effects provided by neighboring structural members. The licensee observed that along the specimen edges, expansion is concentrated in the large crack, whereas away from the edges, expansion is of about the same magnitude but distributed into finer cracks across the specimen cross sections.

The NRC staff specifically noted the consistency between the experimental design and test methods used in the MPR/FSEL LSTP to the database of test data that was used to develop the ACI 318 code (as well as the ASME Code) equations for concrete shear capacity (Report by ACI-ASCE [American Society of Civil Engineers] Committee 326 (Reference 10)) and reinforcement anchorage and lap splice capacity (realistic beam splice specimen as illustrated in Figure 1.6(d) and explained in Section 1.2 of ACI 408R (Reference 11)), and for anchor testing with those provided in response to NRC IE Bulletin 79-02 (Reference 9). This similarity enabled a direct, representative comparison and assessment of the applicability and limitations of the code equations to determine the structural capacity of the range of ASR-affected Seabrook structures for the respective limit states. The NRC staff notes that because the approach for the test programs supplements (rather than replaces) the design code, results from the representative test specimens may be applied for all Seabrook reinforced concrete structures designed using the code. Additionally, the plant-specific features of the MPR/FSEL LSTP further enabled applicability of the test results to the range of Seabrook structures with two-dimensional rebar configurations.

In addition to the review of the LAR, NRC inspectors conducted reactor oversight process inspections of the MPR/FSEL LSTP during implementation to verify that the licensee and its contractors were adhering to the 10 CFR Part 50, Appendix B, quality assurance program requirements and GDC 1. These inspections observed, on a sampling basis, the setup of the program and the facilities, fabrication and concrete pour, and testing of the specimens. The scope and findings of these inspections are documented in NRC Inspection Reports 05000443/2012010, dated August 9, 2013 (Reference 12); 05000443/2013005, dated January 30, 2014 (Reference 13); 05000443/2014002, dated May 16, 2014 (Reference 14); 05000443/2014005, dated February 6, 2015 (Reference 15); and 05000443/2015004, dated February 12, 2016 (Reference 16). During the inspections, the NRC inspectors did not identify any findings related to the MPR/FSEL LSTP and determined that the licensee implemented appropriate quality assurance program requirements.

Based on its review and inspections, the NRC staff finds that the plant-specific MPR/FSEL LSTP was adequately developed and implemented. A detailed discussion of the acceptability of the MPR/FSEL LSTP results and conclusions, and the limitations of applicability of the conclusions to Seabrook structures, is provided below.

3.2.2 Anchor Test Program - Results and Conclusions

Section 5.1, "Anchor Testing," of Report MPR-4273, Revision 0 (Enclosure 6 of Reference 1), summarizes the anchor testing program, which was conducted to quantify the impact of ASR on anchor performance for both post-installed anchors and cast-in-place anchors. This was accomplished by comparing anchor load test results at various levels of ASR expansion to results of tests performed prior to development of ASR, as well as the calculated theoretical failure load. Testing was conducted on two existing girders affected by ASR and **[[ ]]** fabricated test specimens. Testing was performed consistent with the testing used by the vendor for original construction of the plant and as evaluation input for demonstrating compliance with NRC IE Bulletin 79-02, which represents the plant design basis for anchor bolts. Hilti Kwik Bolt 3 expansion anchors were used to represent post-installed, torque-controlled expansion anchors at Seabrook. These were chosen because they are similar to the Kwik Bolt 1 and 2 anchors that have been previously installed at Seabrook. Drillco Maxi-Bolt undercut anchors were used to represent existing cast-in-place anchors and embedment because both anchor types use a positive bearing surface to transfer load to the concrete. A range of anchor sizes and depths was used, and anchors were installed both before and after ASR development in the specimens.

Figure 5-1 (proprietary) of Report MPR-4273, Revision 0, shows the results of the Kwik Bolt 3 (expansion anchor) test and shows that no anchor performance reduction is noted up to in-plane expansion levels of **[[ ]]** millimeters per meter (mm/m). The majority of test results were for in-plane expansion at **[[ ]]** or less on the fabricated block specimens. The observed failure mode was anchor pull-out/pull-through or concrete breakout. Figures 5-2 and 5-3 (both proprietary) of Report MPR-4273 show the results of the Drillco Maxi-bolt (undercut anchor) tests. The results show that no performance reduction was identified until in-plane expansion levels exceeded **[[ ]]** mm/m. Section 5.1.2, "Test Results," of Report MPR-4273 notes that the data in the figures represented anchors installed before and after ASR development, and no significant difference in anchor performance was identified based on installation time. The through-thickness expansion was estimated to vary between **[[ ]]** for the specimens, and the results indicate that anchor performance is not sensitive to through-thickness expansion.

Section 5.4, "Structural Attachments," of Report MPR-4288, Revision 0 (Enclosure 5 of the letter dated August 1, 2016 (Reference 1)), summarizes the conclusions of the anchor testing and the implications for Seabrook structures. Based on the test results, the licensee determined that anchor capacity is not sensitive to through-thickness expansion or time of anchor installation relative to ASR expansion and is not impacted up to a limit of **[[ ]]** in-plane ASR expansion, which was the largest level of expansion seen in the Kwik Bolt tests. This limit is also identified in Table 3.8-18 in the proposed UFSAR markup.

The NRC staff reviewed the information provided on the anchor test program. It was not clear to the staff how the anchor types chosen for the test were representative or bounding of all the anchor types (cast-in-place anchorages and post-installed anchors) used at Seabrook. To gain further understanding of the representativeness of the anchor test program, the NRC staff issued request for additional information (RAI)-T1. In RAI-T1, Request 1, the NRC staff requested the licensee to provide technical justification explaining why the Hilti Kwik Bolt 3 and the Maxi-Bolt post-installed anchors were chosen for testing in the MPR/FSEL anchor test program, as opposed to the other anchor types (manufacturers) installed at Seabrook. In its response to RAI-T1, by letter dated October 3, 2017 (Reference 3), the licensee stated that the Hilti Kwik Bolt 3 and Drillco Maxi-Bolt were selected for testing because they are representative of the load-transfer mechanism of all anchors at Seabrook. The licensee explained that the path through which load is transferred from the anchor to the concrete is the primary consideration for representativeness among anchors, and the selection was informed by industry standards (NUREG/CR-5563, "A Technical Basis for Revision to Anchorage Criteria," dated March 1999; ACI 318; and ACI 349, "Code Requirements for Nuclear Safety-Related concrete Structures and Commentary," 2013) and acceptable practices for comparable evaluations. The anchor size and embedment depth were selected to be consistent with the anchor population at Seabrook.

The licensee also stated that Hilti Kwik Bolt 3 is presently the preferred torque-controlled expansion anchor for Seabrook. It is an updated version of the Kwik Bolt 1, Kwik Bolt Super, and Kwik Bolt 2 anchors that have also been used at Seabrook. Design changes during evolution of the anchor bolt were minor. All of the Hilti Kwik Bolt designs interact with the concrete in the same way and transfer load from the bolt to the concrete using the frictional resistance of the expansion wedge on the concrete.

The licensee further stated that Drillco Maxi-Bolt is the only undercut anchor used at Seabrook. Therefore, there was no need to consider other manufacturers for undercut anchors. An undercut anchor is installed in a special drilled hole in cured concrete. The hole is drilled twice: first, with a conventional drill bit; and second, with an undercutting tool that creates a larger diameter cone-shaped pocket at the desired embedment depth.

The NRC staff reviewed the licensee's response to RAI-T1, Request 1, and finds it acceptable because the anchors tested were selected based on industry standards and accepted practices for comparable evaluations, and the selected anchors represent the load-transfer mechanisms of anchors installed at Seabrook.

In RAI-T1, Request 2, the NRC staff requested the licensee to provide technical justification explaining why cast-in-place anchors (equipment anchors for pumps, motors, etc.) were not included in the MPR/FSEL LSTP and why the test results are applicable to cast-in-place anchors at Seabrook. In its response to RAI-T1, Request 2, by letter dated October 3, 2017, the licensee stated that cast-in-place anchors were not specifically included in the anchor test program because they are represented by the Drillco Maxi-Bolts. The licensee also stated that undercut anchors are similar to cast-in-place anchors, as they both utilize a positive bearing surface to transfer load to the concrete. The installation process for Maxi-Bolts includes use of a special undercutting tool that creates a pocket. When the anchor is set, the expansion sleeve is deployed into the pocket, creating a bearing surface between the sleeve and the undercut hole. This bearing surface is comparable to the interface between a cast-in-place anchor and the concrete that cures around the anchor because both cases rely on a positive bearing surface rather than friction. The licensee explained that at full embedment depth, the load carrying capacity of Maxi-bolt anchors is limited by ductile steel failure as also seen in cast-in-place anchors. However, the test program also included additional tests at reduced embedment depth that produced concrete breakout failures, which provided information on the effect of ASR on concrete breakout mode.

The licensee further stated that cast-in-place anchors may also be able to transfer load through bond between the anchor shank and the surrounding concrete. This extra load-transfer mode is not available to post-installed undercut anchors. Accordingly, Seabrook's approach of using the test results for post-installed undercut anchors to represent cast-in-place anchors is conservative. This evaluation is consistent with the equations in ACI 318 and ACI 349 that allow use of higher adjustment factors for cast-in-place anchors (resulting in higher calculated anchor capacities).

The NRC staff reviewed the licensee's response to RAI-T1, Request 2, and finds it acceptable because both undercut anchors and cast-in-place anchors rely on a bearing surface to carry the applied load. Since the load-transfer mechanism is the same between undercut and cast-in-place anchors, it is reasonable to use undercut anchors in the test program.

The NRC staff reviewed the results of the anchor tests as summarized in Figures 5-1 through 5-3 of Report MPR-4273, Revision 0, and notes that no significant degradation in anchor capacity is identified before **[[ ]]** in-plane expansion. The NRC staff also notes that the results cover anchors installed shortly after specimen casting and shorty before testing. This addresses anchors at Seabrook that may have been installed before ASR development, as well as anchors installed after ASR was identified, up to the identified expansion limit. The NRC staff reviewed the proposed parameter for monitoring (in-plane expansion) and notes that in-plane expansion is a good representation of cracking in the x-y direction. Cracking in the x-y direction is more likely to impact anchor performance because the cracks can lead to a preferential failure path of the anchor bearing surface, which would impact anchor performance. Alternately, through-thickness expansion would capture cracks parallel to the concrete surface. These cracks would be closed by an anchor loaded in tension and the cracks would not provide an additional failure path for the anchor bearing surface. Furthermore, the NRC staff notes that the test results did not show a correlation between anchor performance and through-thickness expansion. Therefore, the NRC staff finds it acceptable for in-plane expansion to be used as the monitoring parameter because it captures cracking in the x-y plane, which could impact anchor performance. Based on its review, the NRC staff finds that the anchor test program provides a reasonable representation of the conditions at Seabrook, and it is reasonable to apply the MPR/FSEL anchor test program results to Seabrook anchors. Therefore, the NRC staff finds that it is acceptable for Seabrook to assume no loss of anchor capacity if in-plane expansion remains below the limit identified in the proposed UFSAR Table 3.8-18 markup, as amended in Enclosure 2 of the letter dated June 7, 2018 (Reference 5).

3.2.3 Shear Test Program - Results and Conclusions

Section 5.2, "Shear Testing," of Report MPR-4273, Revision 0 (Enclosure 6 of Reference 1), summarizes the shear testing program, which was conducted to determine the effect of ASR on out-of-plane shear capacity on reinforced concrete elements without shear reinforcement. Three-point bending tests were conducted on **[[ ]]** deep shear test specimens. **[[ ]]** of these were control specimens that were tested approximately 30 days after fabrication and prior to ASR development, as confirmed by petrography. The remaining specimens were allowed to develop differing levels of ASR, as measured via through-thickness and in-plane expansion, and were tested relative to the performance of the control tests. Two tests were conducted on each specimen for a total of **[[ ]]** shear tests. Figure 5-5 (proprietary) of Report MPR-4273 shows the normalized shear stress-deflection results of the shear tests on all specimens. Consistent with ACI 318, the shear stress was normalized by the square-root of the measured 28-day concrete compressive strength (f'_c), and the shear capacity was defined based on the onset of diagonal cracking. Section 5.2.2, "Test Results," of Report MPR-4273 notes that all of the shear test results exceeded the nominal concrete shear capacity, calculated as $2\sqrt{f'_c}$ per Section 11.4.1 of ACI 318-71, indicating no adverse effect of ASR on shear capacity at the expansion levels tested. Section 5.2.4, "Other Limit States Considered," of Report MPR-4288, Revision 0 (Enclosure 5 of Reference 1), notes that based on a literature review related to two-way shear (i.e., punching shear which involves a truncated pyramid failure surface) and performance of the shear specimens tested for one-way shear (involving a diagonal shear failure plane) in the MPR/FSEL LSTP, punching shear strength of the structural walls and slabs at Seabrook is also not affected by ASR within the expansion limits from the MPR/FSEL LSTP. The licensee's review concluded that ASR had little effect on performance, and the ASR-induced prestress effect appears to counteract any detrimental effects.

Section 2.1, "Structural Limit States," and Section 5.2, "Shear," of Report MPR-4288, Revision 0, summarize the conclusions of the shear testing and the implications on Seabrook structures. The test results demonstrated that no loss of shear capacity (based on $2\sqrt{f'_c}$) was exhibited up to a through-thickness expansion level of **[[ ]]**, which was the highest level of ASR expansion in the shear test specimens. Section 5.2.1, "MPR/FSEL Large-Scale Test Program," of Report MPR-4288, notes that the shear test specimens' expansion behavior was consistent with the reinforcement anchorage specimens in that in-plane expansion levels off at **[[ ]]** and then expansion continues preferentially in the through-thickness direction. Because of this expansion behavior, ASR progression was characterized via through-thickness expansion. The licensee concluded that shear strength (one-way and two-way) of ASR-affected structures can be calculated using the Seabrook design codes, up to the through-thickness expansion limit of **[[ ]]**, provided that ASR expansion behavior is comparable to the test specimens. This limit is also identified in Table 3.8-18 in the proposed UFSAR markup, as amended in Enclosure 2 of the letter dated June 7, 2018 (Reference 5).

The NRC staff reviewed the information provided on the shear test program. The NRC staff notes that all of the shear tests exceeded the nominal concrete shear capacity of the beams (calculated based on $2\sqrt{f'_c}$ in Section 11.4.1 of ACI 318-71), and all of the ASR impacted specimens resulted in shear capacity above that of the control specimens. The NRC staff also notes that the test specimens were more representative (e.g., size, reinforcement detail) of Seabrook structures than existing research, a large number of tests were conducted, and the results were repeatable.

Based on the test design and the consistency of the results, which showed an increase in shear capacity with an increase in ASR, the NRC staff finds it reasonable to conclude that ASR does not adversely impact shear capacity (one-way shear and two-way shear) up to the through-thickness and volumetric expansion limits identified in Table 3.8-18 in the proposed UFSAR markup, as amended in Enclosure 2 of the letter dated June 7, 2018. Additionally, the licensee's implementation of the future confirmatory actions required by the license condition discussed in Section 3.6 of this SE will provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures. Therefore, the NRC staff finds that within the expansion limits, the nominal shear stress, calculated, based on $2\sqrt{f'_c}$ in Section 11.4.1 of ACI 318-71 and the specified concrete compressive strength from the original design, will be bounding for Seabrook ASR-affected structural members.

3.2.4 Reinforcement Anchorage Test Program - Results and Conclusions

Section 5.3, "Reinforcement Anchorage Testing," of Report MPR-4273, Revision 0 (Enclosure 6 of Reference 1), summarizes the reinforcement anchorage test program, which was conducted to determine the effect of ASR on reinforcement anchorage, including lap splices, and on the flexural stiffness of reinforced concrete elements. Four-point bending tests were conducted on **[[ ]]** deep test specimens that contained reinforcement lap splices at the center constant moment region of each beam. One of these was a control specimen that was tested approximately 30 days after fabrication and prior to ASR development (as confirmed by petrography). The remaining specimens were allowed to develop differing levels of ASR, as measured via in-plane and through-thickness expansion, were tested, and the results were compared to the control tests. Figure 5-7 (proprietary) of Report MPR-4273 shows the load-displacement plots for the control test specimen and the test specimen exhibiting the highest level of expansion. Section 5.3.2, "Test Results," of Report MPR-4273 notes that ASR did not result in any adverse effect on the reinforcement anchorage capacity; however, the stiffness behavior was impacted. For all of the ASR-affected specimens, the "yield moment" exceeded the theoretical value (M_y) by **[[ ]]** and the flexural capacity exceeded the nominal capacity (M_n) by **[[ ]]**.

Section 2.1, "Structural Limit States," and Section 5.1, "Flexure," of Report MPR-4288, Revision 0 (Enclosure 5 of Reference 1), summarize the conclusions of the flexure testing and the implications on Seabrook structures. The test results demonstrated that specimens with through-thickness expansion up to **[[ ]]**, which was the highest expansion level exhibited by the test specimens, were able to fully develop the minimum specified lap splice length and exhibited no reduction in flexural capacity. Similar to the shear specimens, in-plane expansion levels off at **[[ ]]** and then expansion continues preferentially in the through-thickness direction. Based on the test results, the licensee concluded that flexural strength of Seabrook ASR-affected structures can be calculated using the Seabrook design codes, up to the through-thickness expansion limit of **[[ ]]**, provided that ASR expansion behavior is comparable to the test specimens. This limit is conservatively identified as **[[ ]]** in Table 3.8-18 in the proposed UFSAR markup (as amended in Enclosure 2 of the letter dated June 7, 2018) to be consistent with the expansion upper limit achieved for the shear testing.

The NRC staff reviewed the information provided on the reinforcement anchorage test program. The staff notes that all of the flexure tests (including control) exceeded the nominal flexural capacity of the beams and all of the ASR impacted specimens demonstrated flexural capacity above the control specimens. Additionally, all of the specimens were able to fully develop the minimum specified lap splice length. The staff also notes that the specimens were more representative (e.g., size, reinforcement detail) of Seabrook structures than existing research, a large number of tests were conducted considering the size of the specimens, and the results were consistent and repeatable. Based on the test design and the consistency of the results, which showed an increase in flexural capacity with an increase in ASR expansion, the NRC staff finds it reasonable to conclude that ASR does not adversely impact flexural and lap-splice capacity up to the through-thickness and volumetric expansion limits identified in Table 3.8-18 in the proposed UFSAR markup, as amended in Enclosure 2 of the letter dated June 7, 2018. Additionally, the licensee's implementation of the future confirmatory actions required by the license condition discussed in Section 3.6 of this SE will provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures. Therefore, the NRC staff finds that within the expansion limits, the flexural strength calculated, based on the ACI code provisions using specified concrete compressive strength from the original design, will be bounding for Seabrook ASR-affected structural members.

3.2.5 <u>ASR Impacts on Other Limit States and Design Implications (Reference 1, Enclosure 5, Sections 5 and 6)</u>

Section 5, "Structural Limit States," and Section 6, "Design Considerations," of Report MPR-4288, Revision 0 (Enclosure 5 of Reference 1), discuss additional limit states and design considerations that may be impacted by ASR. The following SE section addresses the limit states and considerations that are not covered elsewhere in the SE.

Compression Limit State

Section 5.3, "Compression," of Report MPR-4288, Revision 0, discusses the compression limit state and notes that ASR expansion in reinforced concrete imparts an additional compressive stress or load on the concrete (and a corresponding self-equilibrating tensile stress in reinforcement) in directions where expansion is restrained by the reinforcing steel due to an ASR-induced prestressing effect. This ASR-induced compressive load is additive to compressive stresses on concrete due to other design loads; therefore, this additional demand must be accounted for in the design evaluation calculations. This load can be calculated based on the measured in-plane expansion. Apart from this, the licensee's evaluation and literature review concluded that ASR expansion does not reduce the compression capacity of confined concrete in its structural context. Section 5.3 also notes that the results from the MPR/FSEL LSTP flexural testing (discussed previously) provide support for the conclusion that compressive strength of concrete members is not impacted by ASR within the expansion bounds of the test, and that it is acceptable to perform evaluations of impacted structures using the specified nominal compressive strength of concrete in the original design. This is based on the fact that if compression capacity was reduced, a compression zone failure would have occurred in the flexural specimen before the full flexural capacity was realized, and this did not occur in any of the flexural specimens.

The NRC staff reviewed the information provided on the compression limit state and notes that the licensee includes the additional compressive load in the concrete (and associated self-equilibrating tensile stress in the reinforcement) due to ASR expansion in the structural analyses of ASR-affected concrete structures. The NRC staff's review of the analysis method, including how the licensee develops and incorporates the ASR load into the structural evaluation, is discussed in Section 3.3 of this SE. The NRC staff also notes that no compression-controlled failures were identified in the flexural test program, and that all specimens were able to develop the calculated flexural capacity, based on the specified concrete compressive strength, within the ASR expansion limits achieved during the testing. Further, the NRC staff's independent review of the literature related to ASR effects on compression members in the structural context indicates that ASR had no significant effect on bearing capacity of compression members (e.g., Reference 17 (Talley, et al.) states that the ASR/DEF columns had over 1 percent expansion when tested and had no significant reduction in bearing capacity; Section 6.4 of Reference 18 (Blight, et al.) states that compression members are relatively unaffected by alkali-aggregate reaction). These sources provide reasonable assurance that in-situ compressive strength of reinforced concrete members subject to axial compression, or combined axial compression and flexure, is not significantly affected by ASR when expansion remains within the expansion limits determined during the testing and the additional ASR-induced load is accounted for. Thus, based on its review, the NRC staff finds it acceptable for structural design evaluations of ASR-affected Seabrook structures to use the originally specified nominal concrete compressive strength.

Reinforcement Strain

Section 6.1, "Reinforcement Steel Strain," of Report MPR-4288, Revision 0, discusses the possible impact of ASR expansion on reinforcement strain and notes that ACI 318-71 recommends flexural elements be designed such that they are tension-controlled to ensure ductile failure. Tension-controlled elements are designed to allow the reinforcement on the tension side to yield prior to compressive failure of the concrete, which allows for visual indications (e.g., deflections or flexural cracking) of structural distress prior to failure. Section 6.1 further notes that strain beyond reinforcement yield is necessary at failure to ensure a ductile design and, therefore, desirable by ACI 318-71 for ductile design of flexural elements.

The NRC staff reviewed the information provided in Section 6.1 of Report MPR-4288 and notes that the design code allows for reinforcement strains beyond yield for determining the flexural capacity in ultimate strength design philosophy of ACI 318-71 for comparison against ultimate (factored) loads. However, under normal operating or service load conditions, the design code ensures stresses and strains will remain within elastic limits through serviceability considerations, such as crack and deflection control. Seabrook UFSAR Sections 3.8.4.3, "Loads and Loading Combinations," and 3.8.4.5, "Structural Acceptance Criteria," provide definitions and structural acceptance criteria, respectively, of normal operating (service) load conditions for Seismic Category I structures (other than containment). As required by the structural design in Seabrook UFSAR Sections 3.8.4.3 and 3.8.4.5 (the corresponding UFSAR subsections for containment internal structures are 3.8.3.3 and 3.8.3.5), stresses and strains in the structures shall be maintained within elastic limits under normal operating (service) load conditions.

Unlike other service loads, ASR expansion is a self-straining service load whose progression has potential for straining the reinforcement beyond yield under normal operating conditions. Potential yielding of the rebar due to ASR under service conditions could be indicative of a marked change in the behavioral response of a structure, could impact structural capacity, and can render assumptions of linear-elastic behavior in the structural analyses (including seismic analyses in UFSAR Section 3.7) unjustified. The NRC staff notes that the in-plane expansion levels **[[** **]]** in the MPR/FSEL LSTP shear and reinforcement anchorage test specimens did not result in rebar strain exceeding yield values prior to load testing to failure. Since the testing did not directly address the possibility of rebar strain beyond yield, the proposed method of analyzing ASR-affected structures should address the possibility of rebar yield. However, it was unclear to the NRC staff if the proposed analysis method included steps to verify the concrete and rebar stresses and strains, based on realistic (i.e., actual unfactored loads experienced by the structure) behavior under normal operating conditions (including ASR), would remain within elastic limits as required by the UFSAR and Seabrook design code (ACI 318-71). Therefore, by letter dated October 11, 2017 (Reference 19), the NRC staff issued RAI-D8, requesting the licensee to explain how the proposed method of evaluation for ASR-affected structures (designed using strength design philosophy of ACI 318-71) verifies that the stresses and strains in the concrete and reinforcement remain within elastic limits, based on realistic (unfactored) behavior under normal operating (service) load conditions, including an ASR load.

The NRC staff notes that this issue does not apply to the containment designed per ASME Code, Section III, Division 2, because it follows the working stress design philosophy and limits allowable stresses in reinforcement under service load conditions to 0.5 times the yield stress. The licensee's response to the RAI discusses the proposed analysis methodology; therefore, the NRC staff's review of the RAI response, followup RAI D-11, and the staff's conclusion on reinforcement strain under service conditions is addressed in SE Section 3.3.5 related to the method of evaluation. Based on that review, the NRC staff finds that the proposed method of evaluation provides reasonable assurance that strains in the reinforcement of ASR-affected structures remain within elastic limits under unfactored, normal operating (service) load conditions.

Reinforcement Fracture

Section 6.2, "Reinforcement Fracture," of Report MPR-4288, Revision 0, discusses the possible impact of ASR expansion on reinforcement fracture. Examples of reinforcement fracture have been identified in older Japanese transportation structures impacted by ASR. Based on a review of the available literature associated with the failures, the licensee determined that the brittle reinforcement fractures were observed in bent reinforcement (stirrups or hooks) only and were largely in bend diameters smaller than permitted by current U.S. design codes, including ACI 318-71. The failures were brittle in nature, which indicate a change in mechanical properties in the normally ductile steel reinforcement, and testing concluded that the fractures initiated at compression cracks on the interior portion of the bend.

The licensee noted that bending a reinforcement bar results in elongation of the bar on the outside of the bend and compression on the inside of the bend, and the contact with the bending pin flattens bar deformations, which results in large stress concentrations, leading to a potential for compression cracks that may propagate under ASR expansion and result in brittle fracture. Compression stresses increase as rebar is bent to smaller diameters. FSEL performed bend tests of reinforcing bars bent to the allowable limits of Seabrook design codes and did not see evidence of compression crack formation. The licensee also noted that it did not find any reported operating experience of rebar fracture due to ASR in the United States. The licensee concluded that reinforcement fracture is not a concern for ASR impacted concrete constructed to ACI 318-71 or ASME Code, Section III, Division 2 standards; therefore, reinforcement used at Seabrook is not susceptible to brittle fracture.

The NRC staff reviewed the information provided by the licensee and noted that the international operating experience with reinforcement fracture due to ASR expansion was limited to bars that had been bent beyond the allowable bend diameters provided in ACI 318-71 or ASME Code, Section III, Division 2, and in current design codes. In addition, the failures initiated at locations of compression cracks on the inside bend of the bars. The NRC staff noted that the bend tests conducted by FSEL on reinforcement, with bend diameters allowed by Seabrook design codes, did not detect any significant compression cracking in the reinforcement. Based on this, the licensee concluded that reinforcement fracture due to ASR expansion is not a credible concern when structures are constructed to ACI 318 standards. The NRC staff finds the licensee's conclusion acceptable because there has been no operating experience with reinforcement fracture due to ASR in structures designed to ACI 318-71, and the existing operating experience with reinforcement fracture is associated with older transportation structures in Japan that allowed reinforcement bends beyond the limits allowed by the licensee's design codes of record.

Seismic Response and Flexural Stiffness

Section 6.3, "Seismic Response," of Report MPR-4288, Revision 0, discusses the possible impact of ASR-induced expansion and cracking on the stiffness of Seabrook structures and the associated impact on the seismic response. The licensee notes that, in general, a change in stiffness would modify the deflections of a structure for a given static load and the dynamic response of a structure when subjected to seismic loading. A change in deflection under static loads would be addressed by current monitoring programs; however, a change in the dynamic response of a structure would change the seismic loads, deflections, and in-structure response spectra, and could necessitate an updated seismic design. The seismic analysis and design of Seabrook safety-related structures is described in UFSAR Section 3.7(B). In general, a structure's seismic response is affected by the structural stiffness, and flexural stiffness is most impacted by cracking. Cracking also increases a structure's damping ratio, which reduces its seismic response. Therefore, it is conservative to neglect the impact of ASR cracking on structural damping. Based on this information, the licensee determined that it was appropriate to evaluate the effects of ASR on seismic performance in relation to the effects of ASR on flexural stiffness, which is proportional to the modulus of elasticity and cross-sectional moment of inertia of the member. The licensee noted that the MPR/FSEL LSTP results showed an initial (prior to flexural cracking) flexural stiffness of ASR-affected specimens of approximately **[[ ]]** less than the calculated stiffness, and that of the control specimen; this reduction is attributed to the presence of ASR-induced cracks in the specimen prior to application of load. However, the service level stiffness (from 0 percent to 60 percent of yield moment) of ASR-affected specimens was **[[ ]]** larger than the control specimen, with stiffness generally increasing with expansion; this increase is attributable to ASR-induced prestressing effect.

Section 6.3.5, "ASR Effects on Flexural Stiffness," of Report MPR-4288, Revision 0, discusses the impact of ASR on flexural stiffness and noted that the MPR/FSEL LSTP showed that flexural rigidity increased with ASR cracking after the onset of flexural cracking. From a review of Seabrook structure's natural frequencies, the licensee noted that the smallest frequency is approximately 4 Hertz (Hz). Figure 6-2 in Report MPR-4288 shows the response spectrum for Seabrook and demonstrates that seismic demands decrease for frequencies larger than approximately 3 Hz. The increased rigidity would increase the natural frequency and reduce the demand. Prior to the onset of flexural cracking, the MPR/FSEL LSTP results showed a decrease in flexural rigidity of approximately **[[ ]]**. Changes in stiffness change the natural frequency by a square root relationship; therefore, a **[[ ]]** reduction in nominal rigidity would reduce the natural frequency by approximately **[[ ]]**. A reduction in frequency would increase the seismic response by approximately an equivalent amount. The licensee stated that a **[[ ]]** increase in seismic response of the structure due to ASR effects is well within the normal variation in overall concrete properties, and the licensee noted that ACI 318-95 states that the modulus of elasticity can vary as much as 20 percent around the code-specified values. These uncertainties in material properties are factored into the Seabrook original seismic design by broadening the peaks of the calculated in-structure response by at least 10 percent. Based on this information, the licensee determined that ASR does not have a significant impact on the seismic analyses and response.

The NRC staff reviewed the information on flexural stiffness and seismic response. The NRC staff had several concerns regarding how ASR effects on stiffness were being addressed in the method of evaluation, specifically with regard to implementing cracked section properties in the reanalysis of ASR-affected structures. These concerns related to the implementation of cracked section properties led to RAI-D10, which is discussed in Section 3.3.3 of this SE. This portion of the review focuses on the licensee's conclusion that ASR has no significant impact on the overall seismic analyses and response.

The NRC staff notes that cracked concrete does allow for additional damping. However, the ASR-induced prestressing effect could counteract the increase in damping resulting in an insignificant net effect on damping. Therefore, the NRC staff finds it reasonable to neglect the possible ASR impact on damping. The NRC staff also notes that ASR-expansion and cracking has the largest impact on flexural stiffness. This can be seen by the fact that modern structural design codes (ASCE 43-05 and ACI 318-11) provide reduction factors for flexural stiffness to account for changes due to cracked section properties; however, similar factors are not provided for shear or axial stiffness. This indicates that cracking impacts the flexural stiffness more significantly than the other stiffness types. Therefore, the NRC staff finds it reasonable for the licensee to consider the impact of ASR on the seismic response based on the impact of ASR on flexural stiffness.

The NRC staff reviewed Seabrook UFSAR Section 3.7(B) and noted that the smallest natural frequency of a seismic Category I structure is 4.0 Hz. The NRC staff also notes that the natural frequency of a structure is proportional to the square root of the stiffness to mass ratio. Based on a review of the MPR/FSEL LSTP data, the NRC staff noted that for heavily loaded structures (i.e., members with flexural cracking), the flexural stiffness increased as ASR expansion increased. Figure 6-1 in Report MPR-4288 shows the Seabrook design-basis seismic ground response spectrum and shows that seismic demands decrease for frequencies larger than approximately 3 Hz. Since all the structures at Seabrook have a natural frequency of at least 4 Hz, 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 a negative impact on seismic response for heavily loaded structures. For lightly loaded members (i.e., members with no flexural cracking) the test results showed a decrease in flexural stiffness of approximately **[[ ]]**. Based on the square root relationship between stiffness and natural frequency, this reduction could lead to a **[[ ]]** reduction in natural frequency. The NRC staff notes that concrete is a heterogeneous material with variations in properties, which leads to uncertainties in material properties. To account for these uncertainties, modern concrete design includes inherent conservatisms, which are expanded upon in the Seabrook seismic design by using a 10 percent spectra broadening in the response spectra. Based on these inherent conservatisms, the NRC staff finds it acceptable to assume that a small **[[ ]]** decrease in a structure's natural frequency will not have a significant impact on the seismic response of the structure.

The NRC staff further notes that, typically, the seismic response frequency of nuclear power plant structures also depends on, and may be more controlled by, the in-plane shear stiffness of structural walls in addition to the out-of-plane flexural stiffness. The NRC staff notes that the effect of ASR on the in-plane shear stiffness is expected to be comparable to the effect on flexural stiffness and out-of-plane shear stiffness observed in the MPR/FSEL LSTP (i.e., an increase in stiffness relative to control). The in-plane stiffness (shear, flexure) of a structural wall is significantly higher than the out-of-plane stiffness because of the geometry, and thus the corresponding natural frequency in the in-plane direction will be farther to the right of the peak of the response spectrum with lower seismic demand. Therefore, any further increase in frequency due to ASR effects is expected to also result in a decrease in seismic demand. Further, the NRC staff conducted a review of available literature related to testing of scaled ASR-affected structural shear wall elements under in-plane lateral displacement excursions (lateral cyclic loading) and simultaneous axial load (simulating seismic loads) as reported in Reference 20 (Habibi, et al.). The results from this reference found that factors such as confinement and prestressing of reinforcement due to ASR expansion resulted in the ultimate capacity of the ASR shear wall being higher, but less ductile, than that of the regular shear wall specimen. This is similar to that observed with regard to out-of-plane shear capacity and flexural capacity in the MPR/FSEL LSTP.

Based on its review, the NRC staff finds it reasonable to conclude that ASR does not adversely impact the global seismic analyses and response of Seabrook structures.

3.2.6 <u>Instrumentation Test Program - Results and Conclusions</u>

Section 5.4, "Instrumentation Testing," of Report MPR-4273, Revision 0 (Enclosure 6 of Reference 1), summarizes the instrumentation test program, which was conducted to evaluate the performance of candidate instruments and to select the appropriate instrument for measuring through-thickness expansion of Seabrook structures affected by ASR. Section 5.4 of Report MPR-4273 notes that the program evaluated three candidate instruments, a vibrating wire deformation meter (VWDM) and two extensometers, over approximately a 1-year period of exposure on a representative large-scale beam test specimen, with configuration indicated in Table 3-1 in Report MPR-4273. All of the instruments were installed in concrete after core drilling. The extensometers were installed with **[[ ]]** different gauge lengths, resulting in **[[ ]]** total configurations. In order to provide reference expansion measurements, companion holes were drilled on each side of the instruments. A plate was placed on the opposite face of the beam to serve as a contact point for measurements with a depth gauge.

Section 5.4 of Report MPR-4273, explains that the VWDM consists of a vibrating wire strain gauge in series with a spring. The instrument is installed in the core hole and the hole is filled in with grout. Output from the device is measured using a battery-powered readout device. The first extensometer, a snap ring borehole extensometer (SRBE), uses a spring-loaded ring to affix two anchors in the bore hole, which are connected by a gauge rod. Expansion of the concrete is determined by using a calibrated depth micrometer to measure the distance between the reference surface on the anchor and the end of gauge rod. The second extensometer, a hydraulic borehole extensometer, uses a copper bladder, which is expanded with hydraulic fluid, to affix two anchors in the bore hole. Expansion is measured in the same fashion as the SRBE.

Based on the results of the test program with regard to quality of data, ease of installation, and reliability, the licensee determined that the standard-length SRBE was the best instrument option. Instrument data agreed with the reference data within approximately **[[ ]]** while expansion was below **[[ ]]**, which exceeds the current estimated expansion levels at Seabrook. The other instruments either did not agree with the reference measurements, or failed. None of the SRBEs exhibited reliability problems during the testing, while **[[ ]]** VWDMs stopped working and the hydraulic borehole extensometers showed signs of slippage. Additionally, the VWDMs were much more difficult to install due to the necessity of refilling the volume around the instrument with grout.

The NRC staff reviewed the results of the instrumentation tests as summarized in Section 5.4 of Report MPR-4273. The NRC staff notes that the licensee conducted a reasonable test that investigated three different instruments in multiple configurations. The test results demonstrate that the borehole extensometers performed better than the VWDMs in reliability. Additionally, the borehole extensometers are much easier to install, they directly measure the physical expansion, and they do not rely on additional equipment (e.g., a readout device) to function. Of the two extensometers tested, the data showed that the SRBE was more reliable and provided more accurate results. The NRC staff also notes from information provided in the revised ASR monitoring program submitted as Enclosure 2 of the Seabrook letter dated May 18, 2018 (Reference 21), the SRBE design contains no electronics and does not require field calibration. The NRC staff further notes that in the rare event that an SRBE does fail, Seabrook could install another SRBE nearby and continue expansion monitoring without any significant loss of data. Therefore, based on the test results, the NRC staff finds it acceptable for the licensee to use SRBEs to measure future through-thickness expansion of Seabrook structures.

3.2.7 <u>Methodology for Determination of Through-Wall Expansion to Date at Seabrook</u>

Enclosure 1, Section 3.5.1, "ASR Expansion," of the letter dated August 1, 2016 (Reference 1), notes that through-wall expansion is monitored on Seabrook structures and compared against limits developed based on the MPR/FSEL LSTP results. Once in-plane expansion reaches a predetermined limit, extensometers are installed and through-wall expansion is monitored directly. The through-thickness expansion up to the time of extensometer installation will be estimated using an empirical correlation between normalized elastic modulus and through-thickness expansion developed, based on material property test data at different levels of ASR-expansion in the test specimens from the MPR/FSEL LSTP. Following extensometer installation, the total through-thickness expansion can be determined by adding the extensometer measurement to the expansion at the time of instrument installation.

Report MPR-4153, Revision 2 (Enclosure 5 (proprietary), of the letter dated September 30, 2016 (Reference 2), provides details on the correlation. Section 3, "Determining Pre-Instrument Expansion from Elastic Modulus," of Report MPR-4153, Revision 2, further notes that the licensee determined multiple normalized correlation parameters from the test data, including elastic modulus, compressive strength, and splitting tensile strength of concrete. A normalized property is the ratio of the concrete material property measured at different expansion levels (by testing cores from beam test specimens at the time of load testing) to that measured from cylinders cast at the time of fabrication of the test specimens and tested 28 days after fabrication. The licensee reviewed the MPR/FSEL LSTP data, as well as literature data, and determined that reduction in concrete elastic modulus is more sensitive to ASR development than compressive strength or tensile strength and, therefore, modulus is the best parameter to use to estimate through-thickness ASR expansion. Using the test data shown in Figure 3-3 (proprietary) in Report MPR-4153, the licensee developed a best-fit **[[ ]]** least squares regression equation (shown below as Equation 1 (proprietary)) to correlate normalized modulus and through-thickness expansion. The coefficient of determination (R^2) is **[[ ]]** for the developed equation.

[[ ]] (Equation 1)

Section 3.2.2, "Data from Literature," of Report MPR-4153, Revision 2, compares literature data to this empirical formula and notes that the trend from the literature data compares favorably with the developed formula as can be seen in Figure 3-4 (proprietary) of the report. In its response to RAI-M3 by letter dated October 3, 2017 (Reference 3) (discussed in Section 3.2.8 of this SE), the licensee elaborated that although published data (typically based on unconfined specimens) indicates a comparable trend, basing the relationship on MPR/FSEL LSTP data (Equation 1) is more representative because the data come from cores taken from large-scale test specimens with reinforcement configuration and concrete mix design similar to that of Seabrook, and the test program was conducted under a 10 CFR Part 50, Appendix B, nuclear quality assurance program.

In order to obtain the normalized modulus for application of the correlation equation to determine expansion to date of Seabrook structures, it is necessary to know the original (28-day) modulus of the impacted concrete. Section 3.3, "Establishing Original Elastic Modulus at Seabrook," of Report MPR-4153, Revision 2, provides two approaches for determining the original modulus, noting that concrete material property testing during construction of Seabrook measured only the 28-day compressive strength; elastic modulus was not measured during construction. Approach 1 uses the equation from ACI 318-71 (Section 8.3.1) to estimate the modulus based on the measured 28-day compressive strength. In order to use this approach, original construction records must be available from the area of interest. Approach 2 uses "reference cores" taken from Seabrook structures in areas not impacted by ASR and in the vicinity of the extensometers. In order to use this approach, the licensee would need to demonstrate that the reference cores were representative of original construction concrete and unaffected by ASR. Both approaches can be used, and the approach should be selected, based on the specific considerations of the area being evaluated. Section 4.2, "Uncertainty Considerations," of Report MPR-4153, Revision 2, notes that both approaches to determining the modulus can introduce uncertainty. To address this uncertainty and to add a degree of conservatism, a reduction factor of **[[ ]]** is applied to the "normalized modulus" term of the developed correlation equation (Equation 1 above), and the adjusted correlation is shown as Equation 3 (proprietary) in Report MPR-4153, Revision 2. This reduction factor will increase the calculated expansion and account for the possible variability in determining the current modulus.

The NRC staff reviewed the information in Report MPR-4153, Revision 2. The NRC staff noted that Figure 3-4 in Report MPR-4153, Revision 2, shows elastic modulus and corresponding ASR expansion data from laboratory tests reported in published literature, which indicate a trend similar to the relationship determined from the MPR/FSEL LSTP data. The material property data from the MPR/FSEL LSTP support the conclusion that the elastic modulus is more sensitive to ASR development and expansion than compressive strength or splitting tensile strength. Based on the reviewed data, the NRC staff finds that the normalized elastic modulus is the most reasonable correlating property to use in order to determine to-date through-thickness expansion.

The NRC staff also reviewed the developed **[[ ]]** best-fit least-squares regression curve based on the MPR/FSEL LSTP test data and noted that the coefficient of determination is **[[ ]]**. Based on considerations of statistical measures of goodness of fit, this is a reasonable coefficient of determination because the fitted curve accounts for **[[ ]]** percent of the variance in the data and, therefore, the regression curve shown by Equation 1 in Report MPR-4153, Revision 2 (and above in this SE), is a reasonable correlation to determine the expansion to date of Seabrook structures. However, since the correlation curve is based on the MPR/FSEL LSTP data, as well as similar trends seen in the published literature data (summarized in proprietary Figure 3-4 of Report MPR-4153), which have not been previously corroborated in situ on ASR-affected structures in the field, the NRC staff determined that future confirmatory actions will need to be implemented to provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures. To address this, the NRC staff issued RAI-M3, which is discussed in detail in Section 3.2.8 of this SE. In the response to RAI-M3 (Reference 3), the licensee noted that it will implement a confirmatory corroboration study of the curve on Seabrook structures to provide assurance of the continued applicability of the curve. This commitment is captured by the license condition discussed in Section 3.6 of this SE. Based on the licensee's response to the RAI and the license condition, the NRC staff finds the proposed correlation curve acceptable for estimating to date concrete expansion.

The NRC staff also reviewed the proposed methods for determining the original 28-day concrete modulus of elasticity. Both methods are acceptable approaches for estimating the original modulus; however, as noted by the licensee, both approaches may introduce uncertainties. The NRC staff noted that the licensee proposed a **[[ ]]** modulus reduction factor to account for uncertainty and thus increase the calculated expansion when using the curve. The NRC staff finds that the proposed modulus reduction factor, analogous to the capacity reduction factor concept used in the codes for strength design, provides a reasonable conservatism to address uncertainty regardless of the method used to estimate the original modulus of elasticity. Based on the correlation curve being derived using the plant-specific MPR/FSEL LSTP research data, similar trends in the literature data, the proposed corroboration study (see discussion of RAI-M3 in Section 3.2.8 of this SE and the license condition in SE Section 3.6), and the normalized modulus reduction factor, the NRC staff finds the licensee's proposed method for determining through-thickness expansion to date at a location on Seabrook structures prior to the installation of extensometers to be reasonable and acceptable.

3.2.8 Representativeness of MPR/FSEL LSTP Results to Seabrook

Enclosure 1, Section 3.2, of the letter dated August 1, 2016 (Reference 1), notes that the specimens used in the MPR/FSEL LSTP were structurally representative of concrete used in constructing Seabrook structures. Section 2.4.2, "Representativeness Objectives of Test Programs," of Report MPR-4273, Revision 0 (Enclosure 6 of Reference 1), discusses the steps 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. Section 3.1.1, "General Description," of Report MPR-4273, also notes that the specimens used **[[**

]] and **[[**

]]. Additional details on the test specimens can be found in Table 3-1 of Report MPR-4273, Revision 0.

The NRC staff reviewed the information in Report MPR-4273, Revision 0, and noted that the specimen size, along with the reinforcement details and the concrete mix design, makes the MPR/FSEL LSTP specimens more representative of Seabrook structures than test specimens from existing data available in public literature. However, the NRC staff required additional information on the applicability of the test results to the structures at Seabrook, which led to RAIs issued by letter dated August 4, 2017 (Reference 22), and discussed below.

Enclosure 1, Section 3.2.3, "Summary of ASR Implications for Seabrook Structures," of the letter dated August 1, 2016, notes that adjustments to Seabrook design code methodologies are unnecessary if ASR expansion levels remain below limits established during the MPR/FSEL LSTP. The limits for flexural and shear capacity and reinforcement anchorage performance are based on through-thickness expansion, which was selected as the monitoring parameter based on the performance of the specimens in the MPR/FSEL LSTP. Section 5.1.4, "Conclusion," of Report MPR-4288, notes that "[a] limit on in-plane expansion is not necessary, as expansion [observed in the testing program] is predominately in the through-thickness direction."

This statement in MPR-4288 assumes that progression of ASR expansion in the structures at Seabrook will behave in a similar fashion to the test specimens, although no actions had been proposed or taken to validate or corroborate this assumption on Seabrook structures. The staff notes that ASR is a volumetric expansion phenomenon that can preferentially occur in three orthogonal directions based on relative directional restraint. During testing, the in-plane expansion plateaued, but expansion continued in the through-thickness direction due to a lack of reinforcement and restraint in that direction. In its December 23, 2016, response to license renewal RAI B.2.1.31A-A1 (Reference 23), the licensee noted that a small number of existing monitoring locations at Seabrook exhibit in-plane expansion that exceeds the plateau levels seen in the MPR/FSEL LSTP. Although the beam test specimens were designed to be as representative as practical of Seabrook two-way reinforced structural walls, due to potentially varying restraints and boundary conditions in the field, there is a possibility that similar behavior may not occur in Seabrook structural systems. To address this, the NRC staff issued RAI-M2, which requested additional information regarding the assumption that ASR expansion behavior at Seabrook will be similar to that observed in the MPR/FSEL LSTP and justification for the lack of limits on in-plane and volumetric expansion.

In RAI-M2, Request 1, the NRC staff requested the licensee to explain how the apparent assumption that ASR expansion on Seabrook structures would behave similarly to the test specimen's expansion would be validated or corroborated. In its response to RAI-M2, Request 1, by letter dated October 3, 2017 (Reference 3), the licensee noted that a periodic assessment of expansion behavior will be conducted on ASR-affected Seabrook structures, which will include a review of expansion behavior to verify expansion initially occurs in all directions but becomes preferential in the through-thickness direction. The licensee stated that the first assessment was in progress, and that it will repeat the assessment no later than 2025, and every 10 years thereafter. In addition to the expansion assessment, the licensee will also perform an in-plant corroboration study to check the correlation between elastic modulus and expansion that was developed from the MPR/FSEL LSTP data. This corroboration will provide continued assurance that expansion behavior of Seabrook structures is similar to the test program. The NRC staff reviewed the licensee's response to RAI-M2, Request 1, and finds it acceptable because the licensee will conduct periodic assessments of the ASR expansion behavior of Seabrook structures and a confirmatory corroboration study to provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures (i.e., that Seabrook structures will exhibit similar behavior as the test specimens, and to provide field validation of the correlation equation). If the conclusions are found to no longer be applicable, the licensee will enter the information into its corrective action program and address the issue. The requirement to implement these confirmatory assessments and corroboration study is captured by the license condition discussed in Section 3.6 of this SE. Further discussion of the elastic modulus vs. expansion correlation can be found in Section 3.2.7 of this SE, and additional discussion of the corroboration can be found below in the NRC staff's review of RAI-M3.

In RAI-M2, Request 2, the NRC staff requested justification for not providing a specific limit on in-plane expansion, especially considering the operating experience with locations above the plateau levels seen during testing. In its response to RAI-M2, Request 2, by letter dated October 3, 2017, the licensee noted that volumetric expansion (sum of measured in-plane expansion (CI) in two directions and through-thickness expansion (extensometer measurements)) will be monitored on a 6-month frequency. The limit for volumetric expansion will be **[[ ]]** and corresponds to the maximum volumetric expansion observed on a test specimen from the shear test program, which is more restrictive than the maximum of **[[ ]]** seen in the reinforcement anchorage program. The response also explained that a specific in-plane expansion limit was not necessary because in-plane expansion in each direction (x-y) is a component of the volumetric expansion. The NRC staff reviewed the licensee's response to RAI-M2, Request 2, and finds it acceptable because the licensee will monitor volumetric expansion (at locations with installed extensometers) on a 6-month basis, which includes the in-plane expansion components and provides a measure of the level of ASR in the component. The licensee also incorporated reasonable volumetric expansion acceptance criteria into its monitoring program along with the existing limits for through-thickness expansion. The 6-month monitoring interval is conservative, given the confirmed slow nature of ASR progression and any associated structural degradation.

In RAI-M2, Request 3, the NRC staff requested information on how it was determined that areas at Seabrook exceeding the expansion seen during testing are bound by the test results. In its response to RAI-M2, Request 3, by letter dated October 3, 2017, the licensee noted that the maximum volumetric expansion observed to date at Seabrook is **[[ ]]**, which is

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below the **[[** **]]** limit based on MPR/FSEL LSTP results. The highest in-plane expansion (CCI) value is 0.248 percent, which is slightly higher than the highest value of **[[** **]]** seen during the MPR/FSEL LSTP. The licensee explained that it is reasonable for in-plane expansion at the plant to exceed the plateau level seen during the testing because there are many more data points at the plant. In addition, the in-plane expansion seen in the MPR/FSEL LSTP was measured prior to external loads being applied, and, therefore, represented only the expansion due to ASR. The Seabrook structures experience external (service) loads (including the load from ASR expansion in concrete backfill) in addition to ASR, which may increase the CCI measurements and the apparent in-plane expansion. The in-plane expansion in the MPR/FSEL LSTP plateaued in the range **[[** **]]**; the average value was **[[** **]]**. The average present CCI value for locations with extensometers at Seabrook is 0.132 percent; therefore, in-plane expansion at Seabrook is presently consistent with that observed in the MPR/FSEL LSTP specimens. The licensee further explained that a limit on in-plane expansion is not necessary because it is a component of the volumetric limit, and a periodic assessment of expansion behavior will be conducted to ensure that in-plane expansion is plateauing in a similar fashion as was seen during the MPR/FSEL LSTP. If the expansion assessment indicates overall expansion behavior of Seabrook structures is not following that seen during the testing, corrective actions will be taken. The NRC staff reviewed the licensee's response to RAI-M2, Request 3, and noted that the licensee intends to monitor volumetric expansion in accordance with the procedure outlined in Appendix B of Report MPR-4273, Revision 0. In addition, the licensee proposed a conservative volumetric expansion limit based on the test results. Finally, the licensee indicated that expansion behavior will be periodically evaluated to ensure that Seabrook structures are expanding in a similar fashion as the test specimens (i.e., in-plane expansion plateaus at a relatively low value and expansion continues preferentially in the through-wall direction). The NRC staff noted that there are monitored locations at Seabrook demonstrating slightly higher in-plane expansion; however, this result is reasonable because Seabrook structures experience external loads other than ASR, which may increase CCI and apparent expansion. Although the initial behavior assessment is still ongoing, the results to-date do not indicate that the Seabrook structures are behaving differently than the test specimens. The NRC staff also noted that the licensee updated the structures monitoring program (SMP) and UFSAR markup Table 3.8-18 (as amended by letter dated October 3, 2017) to include the volumetric limits. Although the licensee is not proposing an additional limit on in-plane expansion, the volumetric and through-wall limits, paired with the expansion evaluation, provide reasonable assurance that the monitoring program will identify excessive expansion levels, regardless of expansion direction, before the limits are reached.

The NRC staff finds the licensee's response to RAI-M2 acceptable because the licensee proposed a reasonable volumetric expansion limit, which provides a measure of the level of ASR in the component based on the test results, and will periodically evaluate Seabrook expansion behavior to confirm that it is similar to the expansion behavior seen during the MPR/FSEL LSTP. Actions to complete these confirmatory periodic assessments are captured as part of the license condition discussed in Section 3.6 of this SE.

To gain additional information on how the modulus of elasticity vs. expansion correlation developed during the MPR/FSEL LSTP would be corroborated to the structures at Seabrook, the NRC staff issued RAI-M3. In its response to RAI-M3, Request 1, by letter dated October 3, 2017, which asked how it will be determined that data taken from Seabrook matches the correlation curve, the licensee noted that a corroboration study will be conducted using expansion data from the plant. After sufficient through-thickness expansion has occurred since extensometer installation, cores will be taken near 20 percent of the extensometers and the

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normalized elastic modulus will be determined. The correlation will then be used to estimate the expansion, which will be compared to the measured extensometer expansion (pre-instrument expansion from correlation curve plus extensometer reading) to assess degree of agreement. The response further noted that the methodology in MPR-4153 includes an adjusted correlation curve with **[[ ]]** reduction applied to the normalized elastic modulus term of the best-fit curve to account for uncertainty. This is conservative, because it drives the predicted expansion higher. The corroboration study will use both the adjusted correlation and best fit correlation to define the acceptance criterion for successful corroboration. Appendix B of the RAI-M3 response defines the acceptance criterion and provides further detail on how the licensee will determine if the estimated expansion value at the time of corroboration correlates with the measured value. Appendix B explains that the corroboration study will analyze the estimated and measured expansion data in two ways (Test 1 and Test 2) to enable assessment of the data obtained at the time of study and data obtained at the time the extensometer was installed. Test 1 compares the estimated expansion using the best-fit curve at the time of the study to the measured expansion using the extensometer reading, plus pre-instrument expansion based on the adjusted curve. This test is successful if the best-estimate value is less than or equal to the measured value. Test 2 compares the estimated expansion at the time of study (adjusted curve minus the extensometer reading) to the original pre-instrument expansion at the time of installation from the best-fit curve. Test 2 is successful if the original best-estimate pre-instrument expansion value is less than or equal to that estimated at the time of study using the adjusted curve and the measured extensometer reading. Appendix B of the RAI-M3 response states that the corroboration would be considered unsuccessful for a particular location if either test fails. Extensometer locations that fail the corroboration criteria will be evaluated for implications on the correlation curve and conservatism in the methodology, and adjustments to the normalized modulus reduction factor may be made if necessary.

The NRC staff reviewed the information in Appendix B of the response and notes that Test 1 confirms that the correlation does not over-estimate expansion and Test 2 confirms that the correlation does not under-estimate expansion. Using both tests together provides a reasonable range of acceptable elastic modulus values for the corroboration study, which will help confirm that the structures at Seabrook continue to behave in a similar fashion as the test specimens. Since the licensee will conduct a study to confirm that the empirical correlation curve reasonably reflects the through-thickness expansion behavior of Seabrook structures, and the procedure for the study has been clearly defined, along with clear acceptance criteria, the NRC staff finds the licensee's response to RAI-M3, Request 1, acceptable.

In RAI-M3, Request 2, the NRC staff requested that the licensee provide additional information on how the locations for the corroboration study would be determined. In its response, by letter dated October 3, 2017, the licensee stated that the corroboration study would occur at 20 percent of the extensometer (Tier 3) locations, which at the time of the response corresponded to 8 of 38 locations. If additional extensometers are installed in the future, additional locations may be necessary to continue to satisfy the 20 percent requirement. The licensee noted that the 20 percent sample size was consistent with typical sampling rates identified in NUREG-1801, Revision 2, "NRC Generic Aging Lessons Learned (GALL) Report" (Reference 24). The samples will be selected from locations that have experienced at least 0.1 percent measured expansion since extensometer installation, and over the range of best-estimate expansion values on the correlation curve observed at Seabrook at the time of the study. The NRC staff reviewed the licensee's response to RAI-M3, Request 2, and finds it acceptable because the licensee identified a reasonable sample size based on existing NRC guidance, and noted that samples would be distributed along the correlation curve, ensuring that the curve would be corroborated at different levels of estimated expansion.

In RAI-M3, Request 3, the NRC staff requested that the licensee justify that the timing of the corroboration activity, and the number of times the activity will be performed, is sufficient to demonstrate that an adequate validation of the curve exists and will be ensured throughout the life of the plant. In its response to RAI-M3, Request 3, by letter dated October 3, 2017, the licensee stated that the initial study will be performed no later than 2025, and if license renewal is approved, a subsequent study 10 years later. This timeline is selected to provide enough time for a noticeable change to occur in through-thickness expansion between initial extensometer installation and the subsequent studies. The licensee noted that there is a chance that there may not be enough locations in 2025 with differential expansion of 0.1 percent, or that the available locations may not sufficiently cover the range of the correlation. If this occurs, the study will still be performed with the best available data. If enough data do not exist for the subsequent study, the licensee will evaluate the need to repeat or augment the followup study when the selection criteria are met. The NRC staff reviewed the licensee's response to RAI-M3, Request 3, and finds it acceptable, because the licensee will perform the study at a point when expansion values have changed enough that the study will provide meaningful results. If Seabrook expansion never reaches an appropriate level, the study will still be conducted no later than 2025 with the best available data. This ensures that the study will be conducted to corroborate the curve at some point in the future, regardless of the expansion behavior. Based on the slow rate of expansion at Seabrook to date, the proposed 2025 date is reasonable.

Based on its review, the NRC staff finds the licensee's response to RAI-M3 acceptable because the licensee proposed a corroboration study to confirm the correlation curve. The proposed study includes an adequate number of samples and will be conducted at appropriate periods to confirm the curve at least two times over the operating life of the plant. Actions to complete this corroboration study are captured as part of the license condition discussed in Section 3.6 of this SE.

The NRC staff reviewed Report MPR-4273, Revision 0, and noted that Section 3.1.1 states that the concrete mix design for the MPR/FSEL LSTP specimens was specifically designed to accelerate ASR development. This allowed levels of ASR to develop beyond that seen at Seabrook, in sufficient time available for the conduct of the test program (i.e., maximum of 2.5 years for the MPR/FSEL LSTP). Enclosure 1, Section 2.1 of the letter dated August 1, 2016, states that a root cause investigation into ASR at Seabrook concluded that the original concrete mix designs used a slow reacting, coarse aggregate that was susceptible to ASR; however, the application does not discuss the potential influence, with respect to structural effects, of the use of significantly accelerated development of ASR in the large-scale test specimens versus the slow natural development of ASR over time in Seabrook structures. The development of creep effects in concrete depends on the time to loading following the concrete pour; the larger the elapsed time, the smaller the creep effects will be. The development of ASR internal (prestress) load during the early age of concrete following casting of the test specimens could result in ASR-induced in-plane creep effects in the test specimens that counteracts and, therefore, could reduce the measured in-plane ASR expansion effects. This early age creep phenomenon in test specimens is potentially unconservative and is not likely to occur in the normal slow development of ASR where the internal ASR (prestress) load develops a long time after concrete has set. To address this concern, the NRC staff issued RAI-T2.

In RAI-T2, Request 1, the NRC staff requested that the licensee explain how it was determined that the MPR/FSEL LSTP results from specimens with accelerated ASR are not unconservative compared to Seabrook structures with normal, slow ASR development. In its response to RAI-T2, Request 1, by letter dated October 3, 2017, the licensee noted that accelerated ASR development is an approach that has been used by many ASR research programs to investigate ASR impacted concrete. Reputable laboratories, including the National Institute of Standards and Technology and Oak Ridge National Laboratory, are conducting ASR research with accelerated ASR specimens. Based on the results of other research, there is no reason to expect that the test results from the MPR/FSEL LSTP would be compromised due to accelerated ASR development. In addition, the licensee noted that there are plans in place to corroborate and assess the expansion of Seabrook structures to verify that the behavior of the Seabrook structures and the MPR/FSEL LSTP test specimens is similar. The NRC staff reviewed the licensee's response to RAI-T2, Request 1, and reviewed literature associated with ASR research. The NRC staff noted that the vast majority of existing ASR research relies on accelerated ASR development and no significant concerns have been identified related to the acceleration or the validity of the results. More importantly, the NRC staff noted that the licensee has programs in place to confirm that the MPR/FSEL LSTP expansion results align with the ongoing expansion of Seabrook structures. Therefore, the NRC staff finds the licensee's response to RAI-T2, Request 1, acceptable, because the licensee has a confirmatory program in place to provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures, which is captured in the license condition discussed in Section 3.6 of this SE.

In RAI-T2, Request 2, the NRC staff requested the licensee to explain how the possible early age concrete creep effects due to accelerated ASR-induced (prestress) load were accounted for in the MPR/FSEL LSTP, or in the application of the results to Seabrook structures. In its response to RAI-T2, Request 2, by letter dated October 3, 2017, the licensee noted that the MPR/FSEL LSTP does not explicitly address early-age concrete creep effects, but that the approach of monitoring ASR progression via expansion inherently accounts for creep, because measuring expansion includes the impacts of creep and ASR-induced prestressing. In addition, the licensee noted that petrographic examination of the test specimens 28 days after placement did not indicate any concrete distress. This implies that ASR prestress had not applied a load to the concrete at this early stage, when it was most susceptible to creep effects. The licensee also provided a quantitative example demonstrating that the creep in the specimens would be approximately the same as the creep in the Seabrook structures. Based on the observed in-plane expansion, the licensee calculated a tensile load in the rebar of the shear test specimens of **[[ ]]** pounds-force (kip) which translated to a compressive stress of **[[ ]]** pounds per square inch (psi) in the concrete. This value is small compared to the average 28-day strength of **[[ ]]** psi. The licensee then estimated the creep based on a standard industry method and identified **[[ ]]** mm/m of creep, which is small compared to the total expansion of **[[ ]]** mm/m. Using the same method, the licensee estimated the creep in a typical Seabrook structure and determined the creep would be **[[ ]]** mm/m, which is comparable to the creep calculated for the laboratory specimens and small compared to measured in-plane expansion. The NRC staff reviewed the licensee's response to RAI-T2, Request 2, and noted that the test concrete did not show signs of distress after 28 days. This indicates that the concrete was not significantly loaded by ASR expansion during the hydration period when it would be most vulnerable to creep. The NRC staff also noted that the quantitative comparison indicated that the creep in the test specimens and the Seabrook structures would be similar, and in both cases minor compared to the overall expansion. Based on this review, the NRC staff finds the licensee's response to RAI-T2, Request 2, acceptable, because it demonstrates that the creep effect is relatively minor and similar for both the MPR/FSEL LSTP specimens and the Seabrook structures.

Based on its review of RAI-T2, the NRC staff finds that although not explicitly addressed in the MPR/FSEL LSTP, the effects of accelerated ASR development, and the possible early age creep effects, do not impact the test results in a nonconservative manner.

Based on its review of the LAR, including Report MPR-4288 and Report MPR-4273, along with the associated RAIs discussed above (RAI-M2, RAI-M3, RAI-T1, and RAI-T2) and the described future confirmatory expansion assessments and corroboration activities that provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures, as required by the license condition discussed in Section 3.6 of this SE, the NRC staff finds that it is reasonable to apply the conclusions of the MPR/FSEL LSTP to the structures at Seabrook as outlined in the LAR. Based on this, the NRC staff finds it acceptable for the licensee to calculate concrete flexural strength capacity and shear capacity in accordance with the Seabrook design codes (ACI 318-71 or ASME Code, Section III, Division 2, 1975 Edition) provided that the measured through-thickness expansion and volumetric expansion remains below the limits identified in UFSAR markup Table 3.8-18, as amended in Enclosure 2 of the letter dated June 7, 2018.

3.2.9 Independent Internal Peer Review of MPR/FSEL LSTP

As part of its review, the NRC Office of Nuclear Reactor Regulation (NRC/NRR) requested an independent internal peer review by cognizant staff in the NRC Office of Nuclear Regulatory Research (NRC/RES) of the MPR/FSEL LSTP, focused on the overall adequacy of the test program and the conclusions reached by the licensee based on the test program. The NRC/RES staff selectively reviewed the licensee submittals in Report MPR-4153, Revision 2 (Enclosure 5 of Reference 2); Report MPR-4273, Revision 0 (Enclosure 6 of Reference 1); and Report MPR-4288, Revision 0 (Enclosure 5 of Reference 1). The results of this independent internal peer review by NRC/RES staff is documented in an e-mail (with attachments) dated February 23, 2018 (Reference 25). The NRC/NRR staff incorporated the results of the NRC/RES review into the review and conclusions of this SE. In summary, the independent review concurred with the licensee's approaches in general and highlighted the following regarding the reports:

- MPR-4153: There appears to be a good relationship between the concrete expansions due to ASR and the elastic modulus as shown in the MPR/FSEL LSTP and in published literature, with some limited scatter in trend. This does not seem to be the case for compressive and splitting tensile strength where a similar relationship is not readily apparent. It is clear from the testing that relating the elastic modulus with ASR expansion is the preferred option for analyzing Seabrook structures. However, NRC/RES noted that the licensee should corroborate the normalized elastic modulus/expansion curve on structures at Seabrook. This issue is addressed in detail in Section 3.2.8 of this SE.

- MPR-4273: The MPR/FSEL LSTP with the use of large specimens is appropriate, greatly minimizes uncertainties associated with scaling, and enables the licensee to apply the test results to the analysis of the ASR condition existing at Seabrook. Importantly, the sizes of the specimens are of the same order as those at Seabrook and equipped with similar reinforcement. The licensee correctly concludes that evaluations of ASR concrete need to place it in its right structural context because of the confinement effects of reinforcement on ASR expansion.

- MPR-4288: NRC/RES agrees with the overall conclusions of the assessment, noting that the ASR loads (expansion behavior in Seabrook structures) should be consistent with the conditions in the supporting testing program at FSEL. The assessment should also use the applicable limit states in the design code in the same manner as used in the comparison of test results against the code equations. The application of the design equations for the load combinations that include ASR loads also should be consistent with the comparisons of the testing results with code provisions for calculation of the limit state capacities.

3.2.10 NRC Staff Conclusion on MPR/FSEL LSTP and Proposed Expansion Limits

The NRC staff notes from Report MPR-4153, Revision 2, submitted in the letter dated September 30, 2016 (Reference 2), that results of material property testing of cores removed from the MPR/FSEL LSTP beam test specimens, prior to load testing at different ASR expansion levels, show a reduction in concrete material properties (elastic modulus, compressive strength, tensile strength) compared to the 28-day properties. However, as stated in SE Sections 3.2.3 and 3.2.4, the load test results of these ASR-affected beam specimens showed that there was no reduction in structural capacity or performance for the limit states and expansion levels tested. This is because the interaction of the concrete and steel reinforcement subject to ASR expansion was preserved in the large-scale beam test specimens; however, this in-situ structural context (confining effect of reinforcement from interaction between concrete and reinforcement) is lost when a core is removed from a test specimen. The NRC staff further notes from this significant observation from the MPR/FSEL LSTP results, that ASR has a much more detrimental effect on the mechanical properties of concrete cores or cylinders than on the structural behavior or performance of reinforced concrete components (e.g., beams), and this has also been previously noted in literature (e.g., Fan, et al. (Reference 26), Blight, et al. (Reference 18)). Thus, the NRC staff concludes that the MPR/FSEL LSTP test specimens provide a more realistic representation of the in-situ structural behavior of ASR-affected reinforced concrete components than concrete cores.

Based on its review of the application, the NRC staff finds that the licensee developed a representative test program and that it is reasonable to apply the conclusions of the MPR/FSEL LSTP to the structures at Seabrook within the bounds and limits of the test program, regardless of the results of material property testing on ASR-affected concrete cores. This includes using the correlation curve to determine pre-instrument through-thickness expansion, as described in MPR-4153, and using nominal specified concrete compressive strength and specified minimum yield strength of reinforcement from the original design for concrete strength capacity calculations. The finding also includes using the design strength for anchor bolts and using the Seabrook design codes to calculate concrete flexural strength capacity and shear capacity, provided that through-thickness and volumetric expansion remain below the limits in UFSAR markup Table 3.8-18 in Enclosure 2 of the letter dated June 7, 2018. However, since this is a first-of-a-kind approach, the NRC staff determined that a license condition (discussed in Section 3.6 of this SE) was appropriate to require the licensee to implement actions to periodically confirm the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures. Specifically, the license condition requires corroboration of the modulus-expansion correlation developed based on the MPR/FSEL LSTP and assessments of the Seabrook expansion behavior compared to the test program.

The NRC staff evaluations of the licensee's monitoring program and proposed ASR behavior assessment/corroboration actions are documented in SE Sections 3.4 and 3.2.8, respectively.

3.3 Proposed Method of Evaluation for ASR-Affected Structures

Enclosure 1, Section 3.3, "Building Deformation Assessment" (Reference 1), notes that in addition to an internal prestressing effect, ASR expansion can lead to building deformation that, when restrained, results in load and additional stresses on affected structures. This deformation must be quantified and the associated loads must be calculated. The unreinforced concrete fill at Seabrook is also susceptible to ASR expansion and can potentially apply an external load on an adjacent structure. The LAR explains that field data is used to estimate demands on a structure caused by self-straining ASR loads. Once the ASR load is estimated, an appropriate load factor is applied and the ASR load is added to the original design load combinations. The resulting demand is compared to the original design capacity of the structure, assuming original design material properties. After analyzing the structure, a threshold factor is determined for each structure, which quantifies the remaining margin between the factored load, including ASR, and the design acceptance limit. A set of monitoring parameters with corresponding threshold limits are also determined for each structure, which include quantifiable behaviors (strain measurements (e.g., CI), deformation measurements, seismic gap measurements, etc.) that are periodically monitored at specific locations to ensure an ASR-affected structure continues to meet the design acceptance criteria in the UFSAR, as amended by this LAR.

The LAR describes a proposed method for quantifying and analyzing the loads imparted on structures affected by ASR. The proposed methodology is a three-stage process that uses more sophisticated analysis methods and additional field data to improve accuracy of results as the stage increases from 1 to 3. In a Stage 1 evaluation, ASR loads are conservatively estimated based on limited available field data. Regions of a structure that exhibit ASR are analyzed for expansion, corresponding to the most severe cracking locations within that region. Structures that do not meet the design code acceptance criteria using the conservative Stage 1 methods, may be evaluated using Stage 2 analyses. In a Stage 2 evaluation, additional inspections and field measurements are taken to more accurately assess the impact of ASR on the structure. A finite element model (FEM) of the structure is created based on design drawings and benchmarked to the original design analysis of the structure with only the current licensing basis loads. The FEM is then calibrated so the deformations and strains due to unfactored sustained loads, and ASR loads, are consistent with field measurements. The calibrated FEM is then used to compute the ASR loads, which are then factored and combined with demands due to original, factored design loads. Structures that do not meet the original design code acceptance criteria using the Stage 2 methods, are evaluated using Stage 3 analyses. In a Stage 3 detailed evaluation, the self-straining structural demands are calculated using the Stage 2 FEM, and structural demands due to design loads are recalculated by applying the design demands to the FEM. Stages 1 and 2 analyses use the methods from the original design analysis, while in a Stage 3 calculation, consideration is given to cracked section properties, self-limiting secondary stresses, and the redistribution of structural demands when sufficient ductility is available. It is noted that the evaluation of a structure may begin at any of Stage 1, Stage 2, or Stage 3 depending on the margins available in the original design to accommodate the ASR load. It is also noted that the licensee's program allows for physical modifications (i.e., retrofit, repair, or shoring) to the structures rather than further evaluation as an option to ensure the structure continues to meet the design acceptance criteria in the UFSAR.

Section 3.3 of Enclosure 1 of the letter dated August 1, 2016 (Reference 1), explains that all three stages of the methodology result in monitoring measurements and locations, along with associated structure-specific threshold monitoring limits that trigger re-evaluation, which are incorporated into the SMP. For Stage 1 and Stage 2, the calculation supplements the original design calculation; for Stage 3 the calculation supersedes the original design calculation. Enclosure 2 of the letter dated September 30, 2016, includes the completed evaluation of the CEB as an example of a Stage 3 analysis to facilitate review of the proposed methodology (note that the CEB evaluation has since been revised).

The NRC staff reviewed the information contained in the letters dated August 1, 2016, and September 30, 2016, as well as the CEB calculation. In order to discuss the calculation and the methodology in detail, and assess the need for additional information, the NRC staff conducted a site audit during the week of June 5, 2017 (Reference 27). Based on the information in the LAR, and insights gained during the site audit, it was unclear to the staff how the methodology described in the LAR could be consistently applied to multiple structures and how similar results could be obtained if different analysts performed the calculations. This was highlighted by the CEB calculation including significant analysis steps (e.g., development of the ASR backfill load, limits on the use of moment redistribution, departures from the code requirements), which were based on engineering judgement. To address the NRC staff's concerns, a public meeting was held with the licensee on August 24, 2017 (Reference 28). During the meeting, the staff outlined its concerns and the licensee noted that they planned to proceed by providing a document describing the analysis methodology in more detail. After the meeting, the NRC staff issued RAIs focusing on the method of evaluation, which the licensee responded to in a letter dated December 11, 2017 (Reference 4). Enclosure 4 to that letter included the Methodology Document (MD), "Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction for Seabrook Station," that details the analysis procedure for structures affected by ASR (note that the MD was later updated by letter dated June 7, 2018 (Reference 5)). Based on the RAI responses, as well as the information previously provided on the docket, the NRC staff identified five focal areas for review in the analysis methodology: (1) development of the ASR load, including the load due to ASR expansion of concrete backfill, (2) the development of load factors for the ASR load, (3) modifications or supplements to the codes of record, (4) determination of the threshold factor and threshold limits, and (5) maintaining structures within the elastic limit under service conditions. All of these topics are discussed in detail below.

3.3.1 <u>Development of ASR Loads, Including ASR Expansion in Concrete Backfill</u>

As noted above, Enclosure 1, Section 3.3 of the letter dated August 1, 2016, explains that ASR expansion can lead to a prestressing effect and building deformation that results in additional stresses on affected structures. In order to determine the effect on the structure, this deformation must be quantified and the associated ASR loads must be calculated. The LAR proposes a three stage analysis approach to develop the ASR loads, with each subsequent stage applying increasingly sophisticated analysis methods and additional field data to refine the evaluation. This analysis approach is outlined in Section 4, of the MD, which includes several criteria to be considered for determining the starting analysis stage of a structure. The criteria are as follows:

1. Structures with simple geometry that permits structural analysis using closed-form solutions and/or simple finite element models;

2. Structures with localized ASR expansion, or ASR expansion affecting the structure as a whole but with only minor indications of distress;

3. Structures with an apparent robust original design leading to a reasonable amount of margin to accommodate ASR demands;

4. Structures that do not exhibit significant signs of distress.

Structural analyses should start at Stage 1 if they meet all four criteria, Stage 2 if two or three of the criteria are met, and Stage 3 if they meet one or none of the criteria.

Section 2.0 of the MD notes that quantitative measurement of ASR in-plane expansion can be made by summation of crack widths or by measurement of change in the distance between two embedded pins (pin-to-pin). The CI involves measurement and summation of crack widths along a set of perpendicular lines on the surface of a concrete element under investigation. The sum of crack widths is normalized by the length of the reference lines to determine the CI in-plane expansion, typically reported in mm/m. The CCI is the weighted average of the CI in the two measured in-plane directions. The MD explains that crack width summation, or the CCI value, can be used to approximate strain in the concrete, because concrete has a low capacity for expansion before cracking. Pin-to-pin distance measurements between two points using a removable strain gage can also be used to determine expansion; however, these more precise measurements are only capable of determining change in expansion after the pins have been installed because it provides change in length measurements between the pins at different times. Other measurements, such as CI or CCI, must be used to determine a "baseline" strain prior to installation of the pins. Section 3.1, of the MD notes that demands associated with ASR are applied to a structure as a strain load based on CI measurements supplemented by pin-to-pin if available.

The NRC staff reviewed the proposed method for determining loads due to ASR deformation. The NRC staff noted that the licensee proposed using CI (or baseline CI supplemented by pin-to-pin expansion measurements) to estimate the ASR strain in a concrete member. The ASR strain simulated in the analysis model is thus based on CI measurements (or baseline CI supplemented by pin-to-pin measurements) on the structure. Due to the low capacity of concrete for expansion prior to cracking, and CI being a standard method widely used in the field to measure in-plane ASR expansion, the NRC staff finds that CI can be used as a reasonable approximation of the in-plane ASR strain in a concrete member. Additional discussion on the adequacy of CI and pin-to-pin expansion measurement as monitoring methods for concrete degradation can be found in Section 3.4.1 of this SE.

Section 4.4.3 of the MD explains how ASR demands are determined for Stage 3 analyses. An FEM is developed based on design drawings and then calibrated so the deformations and strains due to unfactored sustained loads and ASR loads, are consistent with field measurements of in-plane strain. In locations where concrete backfill is adjacent to structural components, the stiffness of the backfill, as well as the possible ASR expansion of the backfill, must be accounted for in the FEM. Section 4.4.3.2 of the MD details how the backfill pressure acting laterally on embedded walls is estimated. The backfill pressure is originally taken as equal to the overburden pressure at the elevation under consideration. This is taken as an approximate upper-limit for the unfactored lateral pressure since once this pressure is reached, further ASR expansion should occur preferentially in the vertical or other transverse directions. After identifying this upper limit, additional steps are taken to see if the value can be reduced based on field data. If structural deformation measurements are available, and the deformation can be determined to be due to backfill expansion, the backfill pressure may be limited to that which would cause the observed deformation. If field observations show no signs of distress, then backfill pressure may be limited to the pressure that would initiate observable distress in the structural member. Section 4.4.4 of the MD notes that the final step in the development of the ASR loads is correlating the analysis model to the field observations. The model is refined until analysis results correlate to field observations for locations and types of distress (e.g., crack type, direction and location of cracking, deformation location and magnitude). Deviations between the model and field observations may be due to incorrect modeling assumptions or incorrect assumptions related to the ASR loads. The ASR loads and assumed backfill pressure may be adjusted to improve the model correlation. The model is considered acceptable when the location of major structural cracks or cracking regions, structural deformation patterns, and relative building movement align between the model predictions and the observed field measurements. Once the ASR loads and the backfill loads are determined, a load factor is applied and the model is reanalyzed with the other design-basis factored loads.

The NRC staff noted that for Stage 1 analyses the ASR loads are conservatively estimated based on strain values measured in the field. Due to the conservatisms associated with a Stage 1 analysis (i.e., structures with no significant signs of distress or only minor ASR expansion, and robust designs with significant margin), the NRC staff finds it reasonable to estimate Stage 1 ASR loads based on available field measurements of CI.

For Stage 2 and Stage 3 analyses, the ASR structural demands are computed by performing finite element analysis of the structure subject to ASR expansion as measured in the field. Assumptions are also made about the magnitude of ASR expansion in the adjacent concrete backfill and its impact on the structure. The NRC staff notes that ASR is a volumetric expansion process, which occurs in all directions unless restrained; therefore, the NRC staff finds it reasonable for the licensee to assume the starting backfill pressure due to ASR expansion on a structure would be limited to the overburden pressure. If the lateral pressure rises to the level of the overburden pressure, it will begin to expand preferentially in that direction. This is a reasonable approach to estimating the impact of the backfill in situations where there is no visual indication of degradation. The staff also finds that it is reasonable for the licensee to adjust the ASR load, and backfill load, within the constraints outlined in Section 4 of the MD, to correlate the FEM to the observed field conditions. Although the staff finds the process as described in the MD reasonable, it is an iterative process that relies on engineering judgement and involves refining the analysis approach based on the stage. To verify the proposed process is reasonable for each stage and can be effectively implemented for each stage, the staff reviewed multiple calculations and discussed the process with the licensee during a site audit the week of March 19, 2018 (Reference 29). The reviewed calculations were sampled from all three stages and were chosen to ensure the NRC staff reviewed the implementation of all the analysis techniques (e.g., moment redistribution) and structures with unique geometry or degradation (e.g., CEB, Fuel Storage Building). Based on the NRC staff's discussion with the licensee, and its detailed review of the completed calculations, the staff determined the licensee was properly implementing the described methodology through all three stages.

During the site visit, the NRC staff reviewed calculation SGH 170443-CA-01, Revision 0, "Evaluation of Electrical Cable Tunnel" (Seabrook FP# 101166) which implements the guidance in the MD for a portion of the Electric Tunnel structure. The calculation determined that the structure (an embedded wall against concrete backfill with no field observed signs of distress) is adequate for operability; however, when applying the procedure outlined in the Reference 4 version of the MD, to account for potential ASR expansion effects of concrete backfill in areas with no observed signs of ASR distress, the structure does not meet the ACI 318-71 code requirements. It appeared that either the structure may need to be modified to meet code requirements, or the MD guidance may need to be revised to more accurately address structures against concrete backfill that show no signs of distress.

To address this issue, the NRC staff issued RAI-D13 requesting the licensee to explain if the MD would be revised based on the Electric Tunnel calculation results, and if so, to provide the revision with an explanation of the technical basis of the changes. The RAI also requested the licensee to clarify whether applicability of the revised proposed methodology is specific to the electrical tunnel structure, or whether it is generically applicable to any structure with embedded walls against concrete backfill with no observed signs of distress.

In its response to RAI-D13 by letter dated June 7, 2018 (Reference 5), the licensee stated that Section 4.4.3.2 of the MD (included as Enclosure 3 of Reference 5) has been revised to provide an alternative approach to evaluate embedded walls, which are expected to first form flexural cracks (ductile behavior) before shear cracks in the in-situ condition under increasing lateral ASR load from the backfill, and currently show no sign of visible structural cracking. Under factored load considerations, due to lower strength reduction factor for shear compared to flexure and higher load factor for ASR compared to non-ASR loads, the controlling demand to capacity ratio may be governed by shear rather than flexure. This alternate method that can be applied to these types of walls, for estimating the lateral pressure induced by ASR expansion of concrete backfill, allows the concrete backfill pressure to be reduced under the following conditions:

a) Limit the pressure to the lower value corresponding to any structural crack initiation (shear or flexure) for the factored load combinations with inclusion of threshold factor.

b) Increase the monitoring frequency (maximum 2-month interval).

c) Design a retrofit or shoring for implementation after observation of any structural crack.

The NRC staff reviewed the licensee's response and noted that ASR growth is a slow, displacement-controlled process and as a wall deforms and cracks some of the pressure induced by concrete backfill pressure may be reduced. Nevertheless, increased inspection and having a designed retrofit, or shoring, provides assurance that any shear behavior can be controlled in a timely manner. Based on its review, the NRC staff finds the licensee's response acceptable because it only applies to embedded walls with no in-situ signs of distress and limits the proposed ASR concrete backfill load reduction to the point at which any structural crack would initiate. Additionally, the proposed approach provides reasonable assurance that any unexpected consequences from the reduction will be mitigated or controlled in a timely manner by the corresponding compensatory actions of increased monitoring at a conservative interval and having a retrofit design ready for implementation. The NRC staff's concern in RAI-D13 is resolved.

Based on its review of the LAR, including the MD, and its review of calculations implementing the methodology, the NRC staff finds the licensee has developed a reasonable approach, primarily based on field measurements and observations, for estimating the loads due to ASR, including those loads due to expansion of concrete backfill.

3.3.2 ASR Load Factors

Enclosure 1, Section 3.3 of the letter dated August 1, 2016, notes that ASR expansion can lead to building deformation that results in additional stresses on affected structures that were not considered in the original design analyses. Section 3.3.4, "Factored Self-Straining Loads," notes that the ASR load needs to be added to the load combinations in the existing UFSAR Tables 3.8-1, 3.8-14, and 3.8-16, and that an appropriate load factor should be applied to the

ASR load for Seabrook structures designed to the ultimate strength design philosophy of ACI 318-71. The ASR load factor was developed to yield reliability index values similar to load factors specified in the ultimate strength design philosophy of the design code (ACI 318-71). The ASR load factors account for uncertainty in ASR expansion by considering the variability in CI measurements from all ASR monitoring grids in Seabrook structures. The letter dated August 1, 2016, further notes that for unusual load combinations, such as tornado wind combinations, all load factors are taken as 1.0, including those for ASR, which is consistent with the current approach in the UFSAR.

Enclosure 4, "SGH [Simpson Gumpertz & Heger Inc.] Report 160268-R-01 Development of ASR Load Factors for Seismic Category I Structures (Including Containment) at Seabrook Station, Seabrook, NH Revision 0 (Seabrook FP#101039)," of the letter dated August 1, 2016, provides additional discussion of how the ASR load factors were developed. Section 1.4.4, "Reliability Index," of the SGH Report 160268-R-01, explains that the reliability index is a statistical metric used to establish the difference between strength and load. A structure with a high reliability index has a low probability of failure. Section 2, "Development of ASR Load factors for Seismic Category I Structures Other Than Containment," of the SGH Report 160268-R-01, notes that the goal when developing the load factors, was to develop factors that maintained the reliability levels consistent with all other load terms in a load combination that where inherent in the original design code (ACI 318-71). The licensee used a methodology that was based on the work of Ellingwood, et al. (Ellingwood), reported in "Development of a Probability Based Load Criterion for American National Standard A58," NBS Special Publication 577, June 1980 (Reference 30). Section 2.2, "Results of Document Review," of the SGH Report 160268-R-01, notes that this methodology is also the basis for current probability-based limit state design requirements in multiple structural design codes, including American Society of Civil Engineers, "Minimum Design Loads for Building and Other Structures" (ASCE/SEI 7-10); American Concrete Institute, "Building Code Requirements for Structural Concrete (ACI 318-11)"; and American Institute of Steel Construction, "Specification for Structural Steel Buildings" (AISC 360-10). Ellingwood determined that the reliability indices for pre-1980's design codes were on average 3.0 for sustained static, 2.5 for wind, and 1.75 for seismic for load combinations containing these loads. The licensee used these target reliabilities to derive appropriate load factors for ASR-induced stress. Reliability is dependent upon the uncertainty in the calculation of loads (demand) and the uncertainty in the calculation of load resistance (capacity). Based on the plant-specific MPR/FSEL LSTP results, the licensee concluded that ASR has no adverse impact on the strength capacity of Seabrook reinforced concrete structures for critical limit states up to the levels of ASR expansion tested. Therefore, within these expansion limits, the inherent uncertainties in capacity do not change from that previously considered in the design load combinations (i.e., the capacity reduction factors in the design code do not change by including ASR). The uncertainty that is addressed here is in the calculation of loads (i.e., ASR-induced stress).

Section 2.3, "Methodology," of the SGH Report 160268-R-01, explains that ASR severity was separated into four severity zones (Table 1 of Reference 1, Enclosure 4), depending on area coverage and on the magnitude of the ASR expansion as determined via CI. A key parameter in deriving the appropriate load factor to maintain the target reliability is the uncertainty associated with the predicted ASR-induced stress state in the reinforced concrete, which is primarily influenced by the variability associated with the ASR expansion measurements. In implementing the Ellingwood methodology, the licensee performed statistical analysis of all CI measurement data from Seabrook ASR-affected structures (as of April 2016; tabulated in Table A1 of Reference 1, Enclosure 4) in each of the identified severity zones, calculating the mean and standard deviation for each severity zone. Section 2.3.2, "Development of ASR Load

Factors," of the SGH Report 160268-R-01, explains that a parameter (k_{ASR}) was defined to represent the ratio of factored ASR demand to total factored demand. This k_{ASR} ratio varies from 0.4 at Zone I (lowest ASR severity; CI less than 0.5 mm/m) to 1.0 at Zone IV (highest ASR severity; CI greater than 2 mm/m). Static load combinations (which target a reliability index of 3.0) generally require higher load factors than wind and seismic load combinations (which target reliability indices of 2.5 and 1.75, respectively). ASR load factors associated with Zone II are lower than those in Zone I; this is because ASR loads in Zone II (CI 0.5 to 1.0 mm/m), as well as Zones III (CI 1.0 to 2.0 mm/m) and IV, have a significantly lower coefficient of variation than those in Zone I. The enclosure explains that regions of a structure with concrete falling into Zone II or higher (i.e., with CI of 0.5 mm/m and higher) have larger ASR demands, but require a smaller ASR load factor to meet the target reliability indices because the ASR variability in these higher zones is lower. Section 2.5, "Summary," of the SGH Report 160268-R-01, notes that the methodology used maintains the reliability inherent in the ACI 318-71 load combinations. The licensee's study determined the load factors associated with the ASR load (S_a) for Seabrook seismic Category I reinforced concrete structures to be, as below (and UFSAR markup Tables 3.8-1, 3.8-14, and 3.8-16 in Attachment 1 of Reference 1, Enclosure 1):

- For structures other than the containment building, use ASR load factor of 2.0 in load combinations with static (sustained) loads, 1.7 for static plus (normal) wind loads, and 1.3 with static plus seismic (operating basis earthquake) loads, and 1.0 for load combinations involving unusual (extreme) loads such as safe shutdown earthquake (SSE), tornado. When ASR strains are greater than 0.05 percent (0.5 mm/m), these ASR load factors may be reduced by 20 percent, but shall not be less than 1.0.

- For the containment building, use an ASR load factor of 1.0 for all load combinations.

The NRC staff reviewed Enclosure 4 of the letter dated August 1, 2016, and noted that the methodology for determining the ASR load factors was in accordance with the Ellingwood methodology, which is the same methodology that has been used as the basis for developing load factors in the limit-state (or ultimate strength) design philosophy in multiple industry consensus standards, including ACI 318, ASCE 7, and AISC 360. In addition, the licensee determined ASR load factors that maintained the same reliability for the overall load combination when the factored ASR load was included. Based on its review, the NRC staff finds the licensee's proposed ASR load factors acceptable because they were developed using the same methodology used to develop load factors in current consensus standards, and the proposed load factors maintain the reliability of the load combinations found in the existing codes of record. The NRC staff finds the ASR load factor of 1.0 for the Seabrook containment building load combinations acceptable because it is consistent with the deterministic working stress design philosophy of the containment building code of record in which loads are best-estimate loads. The NRC staff also finds the 20 percent reduction in load factor (but not less than 1.0) for ASR expansion strains greater than 0.5 percent (0.5 mm/m) acceptable because uncertainty is reduced when expansions are larger, since CI measurement techniques are more accurate for the relatively larger observed crack widths associated with strain levels greater than 0.5 mm/m.

3.3.3 <u>Modifications or Supplements to the Codes of Record</u>

Enclosure 1, Section 3.3.2, "Evaluation of Self-Straining Loads and Deformations for Seismic Category I structures other than Containment," of the letter dated August 1, 2016 (Reference 1) states, in part, that in a Stage 3 analysis, "[t]he structure is evaluated using strength acceptance criteria in ACI 318-71 for reinforced concrete consistent with UFSAR Section 3.8.4.5. In the Stage [3] evaluation, consideration is given to cracked section properties, self-limiting stresses, and the redistribution of structural demands when sufficient ductility is available. The 100-40-40 percent rule in NRC Regulatory Guide 1.92, Revision 3, is used as an alternative to the SRSS [Square Root of Sum of Squares] method for combining three directional seismic loading in the analysis of structures that are deformed by the effects of ASR."

The NRC staff reviewed the information contained in the letter dated August 1, 2016 (Reference 1), as well as the CEB calculation in Enclosure 2 of the letter dated September 30, 2016 (Reference 2), which was submitted as an example of the implementation of the proposed methodology. The NRC staff discussed the calculation and proposed methodology with the licensee during a site audit the week of June 5, 2017. During the site audit, the NRC staff discussed the use of the 100-40-40 method, the development of ASR load factors, the use of moment (demand) redistribution, how the codes of record requirements are met with the proposed methodology, and the methodology in general. Based on the information in Reference 1, and insights gained during the site visit, it was unclear to the NRC staff how the methodology could be consistently implemented and remain within the bounds of the existing codes of record (ACI 318-71 and ASME Code, Section III, Division 2, 1975). To address this concern, the NRC staff issued RAIs (discussed below) related to the proposed methodology and the apparent modifications or supplements to the existing codes of record. The licensee responded to the RAIs in the letter dated December 11, 2017 (Reference 4).

During its review of the CEB calculation, it was unclear to the NRC staff how the licensee was implementing moment redistribution and how the analysis method would be consistent with the existing codes of record. To address this, the NRC staff issued RAI-D3 and RAI-D4 requesting the licensee to:

a. Explain with sufficient technical detail how the proposed moment redistribution approach, as implemented in Revision 0 of the CEB calculation, meets specific requirements of ACI 318-71 that may be applicable. The staff also requested that the licensee provide supporting technical justification for any portions that deviate from the code requirements; and provide the technical basis for concluding that ACI 318-71 covers the use of moment redistribution for structures receiving a Stage 3 analysis.

b. Provide the acceptance criteria, and technical basis for the criteria, for the structural adequacy of a concrete section that develops a plastic hinge.

c. Explain if there is a limit, or criteria, on the amount of moment redistribution allowed in the proposed process and explain the process when moment redistribution does not provide convergence to a valid set of results in all locations.

d. Confirm that the same structural model and boundary conditions are used for all analyses in the sequence. If this was not the case, describe the different models used, and provide the technical basis for using different models, including the validity of superposing results obtained from different models.

In its responses to RAI-D3 and RAI-D4, in Enclosure 1 of the letter dated December 11, 2017 (Reference 4), the licensee stated that Seabrook amended the analysis method to restrict moment redistribution to be in accordance with the provisions of ACI 318-71, Section 8.6. The licensee further stated that it would revise the CEB evaluation to consider cracked section properties instead of the moment redistribution method used in Revision 0 of the CEB calculation.

The NRC staff finds the response acceptable because NextEra amended the method of analysis for ASR-affected structures to limit the use of the moment redistribution method, if used, to be in accordance with the requirements in Section 8.6 of ACI 318-71. In addition, NextEra revised the CEB evaluation to use cracked section properties instead of the moment redistribution method, which the NRC staff verified during a site audit the week of March 19, 2018 (Reference 29). The NRC staff's concerns in RAI-D3 and RAI-D4 regarding implementation of the moment redistribution method are, therefore, resolved.

During its review of the letter dated August 1, 2016, the NRC staff noted that the licensee proposed a change to the licensing basis, permitting use of the 100-40-40 combination method in accordance with RG 1.92, Revision 3. Based on review of the CEB evaluation report, and discussions with the licensee during the June 5-9, 2017, site audit, it was unclear to the staff that the licensee was appropriately applying the guidance in RG 1.92, Revision 3, which identifies that the 100-40-40 spatial combination method is applicable to response spectrum analysis only. The CEB calculation instead used an equivalent static analysis with the 100-40-40 method.

Therefore, the NRC staff issued RAI-D6 requesting the licensee to clarify whether the 100-40-40 method will be implemented in equivalent static analyses for ASR-affected structures. The NRC staff requested that if so, the licensee provide the technical basis for using the method in conjunction with equivalent static analysis.

It was also unclear how the 100-40-40 method was being implemented consistent with RG 1.92, Revision 3, since the UFSAR markup cites the RG statement that it is generally conservative, while the letter dated August 1, 2016, indicated that the use of 100-40-40 is intended to gain margin. Consequently, the NRC staff requested and reviewed, via the online audit portal, sample 100-40-40 calculations prior to the June 5-9, 2017, site audit, and this subject was also discussed during the site audit. Based on its review and the discussions, the NRC staff identified the following concerns with the reviewed sample calculation:

a. The calculation provided a description and two examples of how the 100-40-40 method was applied for combining the three directional responses to determine the maximum expected response for a single load component (e.g., in-plane shear or moment). The NRC staff concluded that for a single load component, the method implemented produces the same maximum response as the RG 1.92, Revision 3, method.

   However, it is not clear how the 100-40-40 method was applied when there was a multiple load interaction effect, such as satisfaction of the axial force plus moment interaction equations used for design of concrete sections.

b. The calculation included two loads, E_o [the seismic inertia force] and H_e [the soil pushing the embedded part of the CEB]. Based on the method of implementing 100-40-40, the combined $E_o + H_e$ in some cases was less than E_o alone. Inherent in a calculation that produces lower responses for the combination of E_o and H_e, compared to E_o alone, is the potential assumption that there is a defined phase relationship between the two loads. This assumption did not appear to be justified in the calculation.

Therefore, the NRC staff issued RAI-D7 requesting the licensee to:

a. Provide an explanation of the procedure for how multiple load components (e.g., axial force and moment) are combined to perform code interaction checks. Include the technical basis for the method's acceptability.

b. Explain, with sufficient technical detail, why the combination of E_o and H_e in some cases is less than E_o alone. If the explanation assumes a phase relationship between E_o and H_e, provide the technical basis for the assumed phase relationship.

In its responses to RAI-D6 and RAI-D7.a by letter dated December 11, 2017, the licensee stated that it amended the analysis method to eliminate the use of the 100-40-40 method as an option for combining the effects of seismic loading in three directions. The licensee stated that, accordingly, it would revise the CEB evaluation to no longer use the 100-40-40 method and instead use the SRSS method with the equivalent static analysis procedure to be consistent with original design calculations performed. The licensee further stated that, for conditions with multiple components (e.g., axial force (P) and moment (M) interaction), the load components are being calculated by the SRSS method. The SRSS calculated positive and negative axial and moment load components will be used for the P-M interaction evaluation. The licensee provided a revised UFSAR markup for Section 1.8 and Section 3.7(8).2.1 to use for the original SRSS methods in Enclosure 3 of the letter dated December 11, 2017.

The NRC staff finds the licensee's response to RAI-D6 and RAI-D7.a, acceptable because: (1) it amended the analysis method in the LAR to eliminate the previously proposed option to use the 100-40-40 method for combining spatial effects of seismic loading, and revised the CEB evaluation to use the SRSS method with equivalent static analysis consistent with the current licensing bases (verified during a site audit the week of March 19, 2018 (Reference 29)); and (2) it appropriately revised the UFSAR markup for Section 1.8 and Section 3.7(8).2.1 to reflect this change. The NRC staff's concerns in RAI-D6 and RAI-D7.a are resolved.

In its response to RAI-D7.b by letter dated December 11, 2017, the licensee stated that the CEB calculation considered the seismic inertia force, E_o, and soil pushing the embedded part of the CEB, H_e, are in-phase; this resulted in maximum base shear and overturning moment since the static equivalent method and the SRSS responses are used. The licensee further explained that the CEB out-of-plane bending response was influenced by the presence of large penetrations, and the location of applied loads including dynamic soil loads. The dynamic soil response and inertial response may, therefore, counteract each other at limited localized locations. However, since the analyses were repeated for all three input seismic motions, including the opposite directions, these localized locations were covered since the results were enveloped.

The NRC staff finds the licensee's response to RAI-D7.b acceptable because: (1) it clarified that E_o and H_e are assumed to be in-phase to maximize the base shear and moment, and (2) noted that localized locations where E_o and H_e are less than E_o alone are enveloped based on consideration of opposing directions (+/-) of seismic forces. The NRC staff's concern in RAI-D7.b is resolved.

RAIs D3, D4, D6 and D7 addressed specific concerns with the implementation of moment redistribution and the 100-40-40 method. Based on its review of the CEB calculation and its site audit on June 5, 2017, the NRC staff also had concerns about the overall implementation of the proposed methodology. To address these concerns about implementing the proposed method of evaluation in a consistent manner, including the Stage 3 analyses, the NRC staff issued RAI-D2 requesting the licensee to provide a detailed explanation of the Stage 3 analysis methods, and clearly identify, with supporting technical bases, any departures (or modifications or supplements) from/to the existing design codes of record analysis methods. In its response to RAI-D2 by letter dated December 11, 2017 (Reference 4), the licensee included the MD (Enclosure 4 of Reference 4), which defines in detail, the analysis and evaluation procedures for implementing all three analyses stages of the methodology. The licensee stated that the MD provides details of structural inspections, modeling, analysis, acceptance criteria, threshold monitoring, and criteria for further analysis or structural modification when threshold monitoring limits are approached. The response identified five deviations considered as "supplements" to the codes of record and their technical bases, which were also included in Section 5.6 of the MD.

The NRC staff reviewed the response and noted that it provided a detailed explanation of the proposed analysis method and identified the deviations from, or supplements to, the codes of record. The NRC staff notes that all of the proposed supplements represent plant-specific departures from, or supplements to, the current licensing basis of Seabrook structures, and should be adequately captured in the UFSAR markup. However, the NRC staff noted that the response to RAI-D2 did not include an updated UFSAR markup. To address this, the staff requested an updated markup via RAI-D14, which is discussed in detail in Section 3.5 of this SE, along with the adequacy of the UFSAR markup in general. The technical adequacy and acceptability of each identified supplement is discussed below (Note: The supplements are listed as shown in the UFSAR markup in Enclosure 2 of Reference 5).

<u>Supplement 1</u> – Consideration of ASR Loads:

> The UFSAR load and load combination Tables 3.8-1, 3.8-14, and 3.8-16 were modified … to consider the ASR load and load factors for calculating the total demands on structures affected by ASR.

The NRC staff notes that Supplement 1 adds the ASR load and associated load factors to the Seabrook UFSAR load combinations. This is necessary because Seabrook seismic Category I structures have been affected by ASR and the existing codes of record (ASME Code, Section III, Division 2, 1975, and ACI 318-71) do not address reinforced concrete affected by ASR. The NRC staff further notes the development and progression of ASR in concrete causes stresses in both reinforcement and concrete.

The NRC staff reviewed this supplement and finds it acceptable because it clearly identifies the additional loads due to ASR, and associated load factors for the different design-basis load combinations, and appropriately incorporates the loads into the appropriate UFSAR markup sections and tables for all seismic Category 1 concrete structures, including containment. The staff evaluation of the technical acceptability of the proposed ASR loads and load factors are documented in Sections 3.3.1 and 3.3.2 of this SE.

Supplement 2 – Code Acceptance Criteria:

> Strength of reinforced concrete sections affected by ASR can be calculated using the Codes of Record (ASME [Section III, Division 2] 1975 and ACI 318-71) and the minimum specified design concrete strength, provided that ASR expansion is within the limits provided in [UFSAR] Table 3.8-18 for through-thickness and volumetric expansion.

The NRC staff notes that based on Supplement 2, the strength capacity (ultimate strength design) or permissible load (working stress design) for strength limit states (flexure, shear, axial compression, axial tension, anchor capacity) of reinforced concrete sections affected by ASR at Seabrook can be calculated using the respective codes of record (working stress design provisions of ASME Section III, Division 2, 1975 for containment; and ultimate strength design provisions of ACI 318-71 for seismic Category I structures other than containment), and the specified minimum concrete compressive strength (f'_c) from the original design. The NRC staff further notes that the technical basis for Supplement 2 is primarily the MPR/FSEL LSTP results, and is supplemented by evaluation of available literature to assess the effects of ASR.

Therefore, based on the NRC staff evaluation of the MPR/FSEL LSTP and its implications in Section 3.2 of this SE, the staff finds that Supplement 2 is acceptable, provided that through-thickness and volumetric expansion remain within the identified MPR/FSEL LSTP limits as stated in UFSAR markup Table 3.8-18 in Enclosure 2 of the letter dated June 7, 2018 (Reference 5), and the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures is confirmed by the licensee's implementation of the future confirmatory actions required by the license condition discussed in Section 3.6 of this SE.

Supplement 3 – Shear-friction capacity for members subjected to net compression:

> The shear-friction capacity for members subjected to net compression can be calculated using procedures defined in Building Code Requirements for Reinforced Concrete (ACI 318-83 Section 11.7).

Supplement 3 notes that shear-friction capacity for members subjected to net compression can be calculated using procedures defined in Section 11.7.7 of the later code edition, ACI 318-83. The licensee explained that the shear-friction capacity defined by ACI 318-71, Section 11.15 does not address members subjected to sustained compression, and noted that provisions for calculation of shear-friction capacity for members subject to sustained (permanent) net compression are provided in multiple later editions of ACI 318 and ACI 349. The licensee noted in its basis that both ACI 318-71 Section 11.15 and ACI 318-83 Section 11.7.5 essentially place the same limits on the maximum nominal shear stress, and also use the same strength reduction factor for shear.

The NRC staff reviewed Supplement 3 and noted that ACI 318-83, Section 11.7 is identical to ACI 349-97, Section 11.7, which is endorsed by the NRC staff in RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)" (Reference 31). However, the NRC staff noted that Supplement 3 only requested use of Section 11.7.7, while Section 11.7.5 of ACI 318-83 was also cited in the technical basis for the supplement. For consistency and completeness, and to ensure all associated or related requirements and provisions are captured when later editions of codes or portions thereof are used, it appeared that ACI 318-83, Section 11.7, should be invoked in its entirety, in lieu of ACI 318-71, Section 11.15. Therefore, the NRC staff issued RAI-D12, which requested the licensee to provide a technical justification for the use of only ACI 318-83, Subsection 11.7.7, in Supplement 3, or update the supplement to include ACI 318-83, Section 11.7, in its entirety.

In its response to RAI-D12 by letter dated June 7, 2018 (Reference 5), the licensee stated that Supplement 3 in Revision 1 of the MD was updated to invoke Section 11.7 of ACI 318-83 in its entirety.

The NRC staff reviewed the licensee's response and notes that Section 5.6, "Supplement to Code of Record Acceptance Criteria," of Enclosure 3, "Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction" (SGH Document No. 170444-MD-01) of the letter dated June 7, 2018, updated Supplement 3 to read as below.

> Supplement 3 – Shear-Friction Capacity for Members Subjected to net Compression: The shear-friction capacity for members including the effect net compression can be calculated using procedures defined in Building Code Requirements for Reinforced Concrete (ACI 318-83 Section 11.7).

The NRC staff further notes that Enclosure 2 of Reference 5 also included an UFSAR markup of all the code supplements, and Revision 1 of the MD as an UFSAR reference. The NRC staff finds the response acceptable because it updated Supplement 3 to invoke Section 11.7 of ACI 318-83 in its entirety, and also incorporated it into the UFSAR markup.

Based on its response to RAI-D12, the NRC staff finds Supplement 3 acceptable because ACI 318-83 Section 11.7 is identical to ACI 349-97 Section 11.7, which is endorsed by the NRC staff in RG 1.142, Revision 2.

<u>Supplements 4 and 5 (evaluated jointly):</u>

<u>Supplement 4</u> – Flexural Cracked Section Properties:

> Reductions of the gross cross-sectional moment of inertia for analysis shall be computed considering the presence of cracking and the prestressing effects of ASR; alternately, 50% of the gross cross-sectional moment of inertia can be used.

Supplement 4, as originally submitted in response to RAI-D2 in the letter dated December 11, 2017, notes that for flexural cracked section properties it is acceptable to calculate the ratio of cracked over uncracked moment of inertia for flexural behavior with ACI 318-71 equation 9-4, or it is acceptable to define the cracked moment of inertia as 50 percent of the gross moment of inertia. The technical basis for Supplement 4 notes that a ratio of 0.5 is consistent with provisions in ACI 318-14, ASCE 43-05, and ASCE 4-16. Additionally, a review of standard Seabrook concrete sections shows the ratio of cracked to uncracked moment of inertia ranges from 16 percent to 47 percent. Based on this, 50 percent is conservative and explicitly calculating the cracked section moment can provide additional benefit, if necessary.

<u>Supplement 5</u> – Axial and Shear Cracked Section Properties:

> Axial and shear cracking reduces the corresponding stiffness of a structural member. The effect of cracking on reducing the axial and shear stiffness of structural components may be considered in analysis.

The technical basis for Supplement 5 notes that once the net tension on a concrete section reaches or exceeds the tensile stress limit of concrete, the stiffness is reduced. In the licensee's analysis, this is done gradually to account for possible aggregate interlock, which is conservative compared to abruptly reducing the tensile strength to zero. The axially cracked section and shear cracked section properties are calculated based on the procedure in Appendix A of the MD.

The NRC staff reviewed Supplements 4 and 5, as well as the information contained in Appendix A of the MD, Revision 0. The NRC staff noted that the proposed approach for determining reduced stiffness for implementing cracked section properties was reasonable for normal, reinforced concrete; however, the approach did not appear to take into account the impact of ASR. Reports MPR-4288 and Report MPR-4273 (Enclosures 5 and 6 of Reference 1) summarize the MPR/FSEL LSTP and the results of the testing appear to indicate that the stiffness of ASR-affected test specimens is higher than the control specimen and show an increasing trend in flexural and shear stiffness and a delay in the onset of flexural cracking, with an increasing level of ASR-expansion (up to the expansion levels tested). This trend is attributable to an ASR-induced prestressing effect. The approach described in Supplements 4 and 5 did not appear to consider the test results. To address this apparent disparity, the NRC staff issued RAI-D10, which requested the licensee to explain how the relevant MPR/FSEL LSTP data pertaining to ASR effects on stiffness was considered in the proposed methodology for determining reduced stiffness (flexural, shear and axial) when implementing cracked section properties.

In its response to RAI-D10 by letter dated June 7, 2018 (Reference 5), the licensee stated it had revised the MD to Revision 1 (included as Enclosure 3 of Reference 5) to modify the cracking moment equation and to clarify the strain definitions for crack initiation for structural members subjected to ASR expansions. The MD, with revised Sections 4.4.5, 5.6 and Appendix A, provides cracked section properties consistent with the stiffness behavior observed in the MPR/FSEL LSTP. The revised equation for cracking moment simulates the observed flexural stiffness increases, which are caused by delayed onset of flexural cracking due to the ASR prestressing effect. The revised MD further clarifies that the tensile and shear crack initiations are based on net concrete strain after overcoming the concrete prestressing effects due to ASR expansion, which is internally included in the finite element model used in the structural analysis. The licensee conducted an assessment and noted that completed structural evaluations using the previous revision of the MD are not impacted by the changes made in Revision 1 of the MD because of one or more of the following reasons:

- No structural cracking was used to reduce member stiffness,
- Stiffness reduction due to cracking (tensile, shear, or flexure) is computed based on concrete strain after overcoming compressional prestraining due to ASR-induced prestressing, or based on field measurements using structural crack widths,
- Flexural stiffness reduction in members is not impacted by ASR expansion because:
    - Members have zero or negligible ASR expansions, or
    - Members are under net tension at flexural cracking locations, or
    - Member stiffness reductions used are confirmed per Revision 1 of the MD

The NRC staff reviewed the licensee's response and notes that, consistent with the wording of revised Supplement 4, Equation 9-5 of ACI 318-71 was modified in Section 4.4.5 of the MD (Revision 1) to account for prestressing effects of ASR in determining flexure cracked section properties. The staff also notes that the alternate provision of using 50 percent of the gross moment of inertia is also supported by the results of the MPR/FSEL LSTP. The staff further notes that the procedure in Appendix A of the MD, to check the onset of shear cracking and axial cracking, captures the effect of overcoming the precompression due to ASR, which is internally included within the FEM and, therefore, the calculated reduction in shear and axial stiffness account for the ASR prestressing effects. Based on its review, the NRC staff finds the licensee's response acceptable because the licensee: (1) appropriately revised the MD to account for ASR prestressing effects, consistent with the MPR/FSEL LSTP results, when determining reduced stiffness for implementing cracked section properties, and (2) provided an adequate rationale from its assessment of calculations already completed using a previous version of the MD to conclude that the changes did not have an adverse effect on these calculations.

Based on its review above, including the response to RAI-D10, the NRC staff finds Supplements 4 and 5 acceptable because the supplements determine reduced stiffness properties (flexure, shear, axial) consistent with industry standards for normal reinforced concrete, with appropriate modifications to account for ASR prestressing effects as observed in the MPR/FSEL LSTP. The NRC staff's concerns in RAI-D10 are resolved.

Based on its review of the five identified code supplements, including the responses to RAIs-D10 and D12 as discussed above, the NRC staff finds that the supplements are technically adequate and properly capture the proposed plant-specific modifications or supplements to the codes of record, and therefore, the response to RAI-D2 is acceptable. The NRC staff's concerns in RAI-D2 regarding departures from the codes of record are, therefore, resolved. Additional discussion about capturing the supplements properly in the UFSAR can be found in Section 3.5 of this SE.

Based on its review of the LAR, including the MD, and RAI responses discussed above (RAIs D2, D3, D4, D6, D7, D10, and D12), the NRC staff finds that the licensee has adequately identified the plant-specific modifications or supplements to the current codes of record for evaluating ASR-affected structures at Seabrook, and has provided a reasonable technical justification for each departure. Therefore, the NRC staff finds the licensee's proposed supplements to the codes of record acceptable on a plant-specific basis, provided that through-thickness and volumetric expansion remain within the identified MPR/FSEL LSTP limits as stated in UFSAR markup Table 3.8-18 in Enclosure 2 of the letter dated June 7, 2018 (Reference 5), and the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures is confirmed by the licensee's implementation of the future confirmatory actions required by the license condition discussed in Section 3.6 of this SE.

3.3.4 Threshold Factor and Threshold Limit

Enclosure 1, Sections 3.3.2 and 3.3.3 of the letter dated August 1, 2016 (Reference 1), discuss a "threshold limit" for monitoring ASR effects for each structure and analysis stage, to define criteria for reevaluation of structures with ASR deformation. The threshold limit is the value for each monitoring element at which the factored (unfactored in case of containment), self-straining ASR load equals the code design limit when combined with the factored (unfactored in case of containment), design-basis loads. In a Stage 1 analysis, an acceptance limit of 90 percent is placed upon the threshold limit. In a Stage 2 analysis, a limit of 95 percent is used, and in a Stage 3 analysis, a limit of 100 percent is used. For Stage 1 and Stage 2 analyses, existing design-basis analysis methods are used, and the threshold limit represents the margin remaining (factor to accommodate future ASR expansion) between the code allowable limits and the design-basis loads, or demands, plus the self-straining loads from ASR. However, in Stage 3, additional analysis methods are employed (e.g., cracked section properties, moment redistribution) that modify structural demands, along with the threshold factor applied to account for future ASR expansion. Section 7.3 of Revision 0 of the CEB evaluation report (Enclosure 2 of the letter dated September 30, 2016 (Reference 2)) stated, in part, "The threshold factor is selected to be the largest factor in which the structure meets evaluation criteria using the approaches described in this calculation," and a threshold factor of 1.2 is reported for the CEB. As discussed in Section 7.6.2 of Revision 0 of the CEB evaluation report, Stage 3 analysis uses an iterative process that allows moments to be redistributed to demonstrate that demands meet code capacities. However, it was not clear if there is a specific limit to the amount of moment redistribution that can be done in the analysis. Since the demands upon the structure are being modified in Stage 3 analyses, it was not clear what exactly the threshold factor represents or how it will be selected in future Stage 3 analyses. It was also unclear if there was a limit placed on the amount the demands could be modified to develop the threshold factor.

To address this, the NRC staff issued RAI-D5, which the licensee responded to in its submittal dated December 11, 2017 (Reference 4). In its response the licensee stated, "the threshold factor is design [engineering] margin expressed as the amount which ASR loads can increase beyond values used in the calculations such that the structure or structural component will still meet the allowable limits of the code of record as supplemented (as discussed in RAI-D2 response)…. It is an outcome of the evaluation, not an input to the analysis methodology approach." The licensee further stated that a unique threshold factor is calculated for each structure based on the available margin and a factor may be revised based on further analysis or by using additional or more refined inspection data. There is no limit on reevaluation, provided the evaluation satisfies the applicable code of record, with the proposed supplements. The response highlighted the fact that moment redistribution will be limited to that allowed by the ACI code of record. If an acceptable threshold factor cannot be developed based on the analysis method in the MD, structural modification may be used to reestablish a margin of safety.

The NRC staff reviewed the licensee's response and noted that the analysis method would follow the codes of record plus the supplements (as discussed in RAI-D2 above, in Section 3.3.3 of this SE) and noted that moment redistribution would be limited to that allowed by the ACI code of record. This makes it clear that for all three stages of the analysis, the threshold factor represents the remaining margin to the code allowable limits that accounts for permissible potential future progression of ASR expansion. In addition, the revision of demands via moment redistribution is limited by the code requirements. Therefore, the NRC staff finds the licensee's response to RAI-D5 acceptable, and finds the threshold limit is a reasonable way to quantify the remaining margin in the structural analyses.

3.3.5 Maintaining Reinforcement Stresses and Strains within Elastic Range under Normal Operating (Service) Load Conditions

The NRC staff notes that in the ultimate strength design philosophy of ACI 318-71, the flexural capacity is determined with tensile reinforcement strains well beyond yield (when concrete is at compressive failure strain) for comparison against ultimate (factored) loads. Further, the staff notes that the fabricated test specimens in the MPR/FSEL LSTP did not develop in-plane ASR expansion to levels that exceeded yield conditions prior to the load test. However, as discussed in Section 3.2.5 of this SE, unlike other service loads, ASR expansion is a self-straining service load whose progression has the potential for straining the reinforcement beyond yield under normal operating or service conditions. Seabrook UFSAR Sections 3.8.4.3 and 3.8.4.5 provide, in part, definitions and structural acceptance criteria, respectively, of normal operating (service) load conditions for seismic Category I structures (other than containment) designed to ACI 318-71 ultimate strength design philosophy. As required by the structural design in the Seabrook UFSAR Sections 3.8.4.3 and 3.8.4.5 (corresponding UFSAR subsections for containment internal structures are 3.8.3.3 and 3.8.3.5), stresses and strains in the structures shall be maintained within elastic limits under normal operating (service) load conditions. Potential yielding of the rebar due to ASR under service conditions could be indicative of a marked change in the behavioral response of a structure, could impact structural capacity, and can render assumptions of linear-elastic behavior in the structural analyses (including seismic analyses in UFSAR Section 3.7) unjustified. However, the proposed method of structural evaluation for these ASR-affected structures (other than containment), which includes provisions for cracked sections and redistribution of structural demand, did not appear to include a verification of the concrete and rebar stresses and strains based on realistic behavior under unfactored, normal operating conditions (including ASR) that would ensure they remain within elastic limits, as required by the UFSAR.

Therefore, the NRC staff issued RAI-D8 requesting the licensee to explain how the proposed method of evaluation (Stage 1, 2 and 3), for ASR-affected structures (other than containment) verifies that the stresses and strains in the concrete and reinforcement remain within elastic limits based on realistic (unfactored) behavior under normal operating (service) load conditions, including ASR load.

In its response to RAI-D8, (Enclosure 1 of Reference 4), the licensee summarized that seismic Category I structures (other than containment) that were designed to ACI 318-71, and that are analyzed using approaches described in the MD (provided in response to RAI-D2) and meet the acceptance criteria therein, will respond elastically under realistic (unfactored) normal operating or service load conditions. The licensee based its conclusions on: (a) two hypothetical parametric studies that examine the effects of increasing ASR expansion coupled with external loads on rebar stress; and (b) a confirmatory evaluation of calculated rebar and concrete stress results for a sample of eight representative Seabrook seismic Category I structures under two normal operating load combinations (LC1 and LC2). The evaluation shows that the maximum rebar stress is below specified minimum yield strength (f_y), and maximum concrete compressive stress and strain are well below the specified minimum compressive strength (f'_c) and usable compressive strain (ε_c).

The two hypothetical parametric studies provide insights on the response of structural members subjected to the combined effect of internal ASR load and external design loadings that are relevant to the behavior of Seabrook structures. Parametric Study 1 evaluated the effects of increasing ASR expansion on rebar stress for a member already loaded with external loadings (sustained or static). The member is first subjected to the combined axial load (P) and bending moment (M) due to external loads, and then the internal ASR load (in-plane expansion) is increased from 0 to 2.0 mm/m. The ASR load simulates the self-straining behavior of placing the steel in tension and concrete in compression. The response results for several cases of P-M combinations, and rebar stress vs ASR expansion results, were provided (as figures and tables) for factored and service (unfactored) load levels. The relevant results are discussed in detail below.

Parametric Study 2 evaluates the effects of increasing external bending moment on rebar stress of a section that already experienced self-straining stresses due to different levels of ASR for the same two member sections as in Study 1. The NRC staff noted that Study 1, and the actual confirmatory evaluations (described below) were more relevant to Seabrook structures.

The results of the confirmatory evaluations for a sample of eight representative Seabrook seismic Category I structures are tabulated in Table 2 and Table 3 of the RAI response. The calculated rebar and concrete stress results under two controlling unfactored (realistic), normal operating (service) load combinations (LC1 and LC2) were provided for the eight Seabrook structures listed below. These structures were qualified by the strength design method of ACI 318-71, as supplemented by the MD, and were justified as being representative of Seabrook ASR-affected structures based on analysis stage, different levels and variations in ASR expansion, varying shape and geometry, and different exposure to concrete backfill. The stresses are calculated using a fiber section method subjected to internal ASR strain and external loads on the section considering linear elastic behavior.

<u>Stage 3 Analysis</u>:
- Control Room Makeup Air Intake structure (CRMAI)
- Residual Heat Removal Equipment Vault structure (RHR)
- Containment Enclosure Building (CEB)

<u>Stage 2 Analysis</u>:
- Enclosure for Condensate Storage Tank (CSTE)
- Main Steam and Feed Water West Pipe Chase and Personnel Hatch (WPC/PH)

<u>Stage 1 Analysis</u>:
- Containment Equipment Hatch Missile Shield structure (CEHMS)
- Containment Enclosure Ventilation Area (CEVA)
- Safety-Related Electrical Duct Banks and Manholes (EMH) W01, W02, W09, and W13 thru W16

The following two load combinations were used to calculate rebar and concrete stresses for (unfactored) normal operating (service) load conditions:

LC1 (in-situ condition): $D + L + E + T_o + S_a$
LC2 (in-situ condition + OBE + future ASR): $D + L + E + T_o + E_o + H_e + F_{THR}{*}S_a$

where D is dead load, L is live load, E is lateral earth pressure, T_o is operating temperature, E_o is the operating basis earthquake (OBE), H_e is dynamic earth pressure due to OBE, S_a is ASR load, and F_{THR} is the threshold factor that accounts for future ASR.

The maximum tensile stress reported for the controlling unfactored service load combination, LC2, is 56.5 kilopounds per square inch (ksi), which occurs in the reinforcement in the East exterior wall of the RHR at the connection to the Primary Auxiliary Building (PAB). This was localized and determined based on conservative modeling. The maximum tensile stress for rebar in other buildings evaluated was less than 45 ksi, which is well below the specified yield stress of 60 ksi. The maximum compressive strain in concrete is 0.00085, which is significantly less than the ACI 318-71 Code maximum usable strain for compression of 0.003. The licensee thus concluded, the rebar stresses are below elastic limits for all structures listed here when considering the two realistic unfactored service level loadings. The concrete compressive stress remains below the crushing limit (4 ksi for CEB and CSTE; 3 ksi for the other structures), and the concrete strains are less than 0.001, which is much less than ACI 318-71 Code maximum usable strain for concrete compression of 0.003. The licensee explained that the structures presented represent the seismic Category I structures at Seabrook that are subjected to ASR expansion and, therefore, the other seismic Category I structures do not require explicit evaluation of stresses for unfactored service level loads. The licensee also concluded that, since the structures meeting the analysis and acceptance criteria described above ensures that the response remains elastic under normal operating or service load conditions, the stress check described in the RAI request does not need to be incorporated into the MD and does not need to be performed for the remaining structures at Seabrook.

Table A: Rebar and Concrete Stress Ratios for Unfactored Service Load Combination, LC2

Structure (from Table 3*, column 1)	**Component** (from Table 3*, column 4)	**ASR load F_{THR} * S_a, mm/m** (from Table 3*, column 3)	**Rebar stress ratio, f_s/f_y** (fs from Table 3*, column 5)	**Concrete stress ratio, f_c/f'_c** (fc from Table 3*, column 6)
CRMAI	Base Mat	1.4 x 0.99 = 1.39	0.65	0.11
RHR	East Exterior Wall	1.2 x 0.75 = 0.90	0.94	0.70
CEB	Wall Near Foundation	1.3 x 0.60 = 0.78	0.71	0.67
CEB	Wall Above Electrical Penetration	1.3 x 0.10 = 0.13	0.93	0.33
CSTE	Tank Wall	1.6 x 0.43 = 0.69	0.45	0.28
WPC/PH	North Wall	1.8 x 0.24 = 0.43	0.74	0.45
CEHMS	East Wing Wall	1.5 x 0.72 = 1.08	0.69	0.51
CEVA	Base Slab	3.0 x 0.31 = 0.93	0.73	0.36
EMH	W13/W15 Walls	0.25 x 3.7 = 0.93	0.45	0.30

f'_c = specified minimum concrete compressive strength (4 ksi for CEB & CSTE; 3 ksi for others)
f_y = specified minimum yield strength of reinforcement = 60 ksi
* Table 3 of RAI-D8 response; Sa = ASR load; F_{THR} = threshold factor

The NRC staff reviewed the licensee's response and notes that LC2 is the controlling unfactored normal (service) load combination since it includes the OBE loads and future ASR expansion (from the current in-situ condition in LC1) as accounted for by the threshold factor. The staff notes from its tabulation of rebar and concrete stress ratios in the above Table A, the maximum rebar stresses are generally below 0.75 times yield stress (with two exceptions with ~0.94 ratio, which remain below the minimum yield strength) and the maximum concrete stresses are generally below 0.5 times the specified minimum compressive strength, f'_c (with two exceptions with ~0.7 ratio, which remains below f'_c). The NRC staff also notes from Table A that the ASR load levels, with the threshold factor included, are below Severity Zone 4 (in-plane expansion greater than 2 mm/m) discussed in the next paragraph. Based on the results tabulated for the eight representative structures using current ASR expansions, the NRC staff finds that the rebar and concrete stresses, under unfactored, normal operating (service) load combinations, are generally expected to be within limits that ensure elastic behavior of the structure under realistic normal operating conditions, including future ASR load accounted for by the threshold factor. Additional discussion on actions that will be taken if it appears elastic limits may be exceeded is provided below in the discussion of RAI-D11.

Additionally, the NRC staff noted that a conclusion of Parametric Study 1 in the response to RAI-D8 states, "Stresses and strains in steel rebar are less than the elastic limits at service load conditions, provided that ASR strain is less than 2 mm/m." This is consistent with the approximate strain level at which rebar is expected to potentially yield (i.e., f_y/E_S = 60 ksi/29000 ksi = 0.0021 mm/mm or 2.1 mm/m). Alternately, ASR expansion exceeding this level could be indicative of potential rebar slip due to loss of bond between concrete and steel reinforcement. Furthermore, ASR in-plane expansion may continue to increase with ASR progression under service conditions and, based on field monitoring, the structural analysis may eventually include the ASR Severity Zone 4 (CI greater than 2 mm/m, as noted in Section 2.3.1 and Table 1 of the SGH Report 160268-R-01 (Enclosure 4 of Reference 1). However, for structures designed to ACI 318-71 ultimate strength design, there is no criteria or upper limit of in-plane expansion in the method of evaluation (i.e., MD) that would trigger an action for evaluation of the implications of potential rebar yielding or rebar slip if cracking levels under service conditions are in Severity Zone 4. Potential yielding or slip of the reinforcement could be indicative of marked change in behavioral response of a structure or component, could impact structural capacity, or could render assumptions of linear elastic behavior in the structural analyses incorrect (including UFSAR Section 3.7 seismic analysis). The NRC staff noted that an action is necessary to evaluate implications of potential rebar yielding (or slip from loss of bond) when ASR progression data indicates the need for the structural analysis to include ASR Severity Zone 4, given that there is no other means of evaluating implications of potential rebar yielding (or slip) in a structure that includes expansion at ASR Severity Zone 4. Therefore, the NRC staff issued followup RAI-D11, requesting the applicant to explain how a structure will be evaluated in the proposed method of evaluation for the implications of potential rebar yielding or slip under service conditions if field monitoring data indicates a structure has entered, or includes, ASR Severity Zone 4 (CI greater than 2 mm/m).

In its response to RAI-D11 by letter dated June 7, 2018 (Reference 5), the licensee stated that Sections 3.1.1 and 3.1.1.1 of the MD have been revised (Revision 1, included as Enclosure 3 of Reference 5) to address actions that should be performed when the CI or CCI value exceeds 2 mm/m (Zone 4) for seismic Category I structures and the containment building, respectively. The NRC staff noted that, if CI or CCI values (in-plane expansion or strain), after adjustment to exclude structural cracks, exceeds 2 mm/m, the revised MD recommends consideration of performing petrography on extracted cores from the area to confirm the status of ASR expansion prior to using these measured in-plane strain values to characterize ASR expansion loading for structural analysis and evaluation. If the CI or CCI values, after adjustment and petrographic confirmation exceeds 2.0 mm/m, then the possibility that local yielding has occurred while the structural response remains within elastic behavior shall be evaluated. If the in-plane ASR strain is confirmed to exceed 2.0 mm/m over a large region, then the MD requires considering retrofit or repair to mitigate the possible reinforcement slippage due to near surface delamination, or further analysis to qualify the structure. The licensee also noted that rebar slippage due to ASR is very unlikely up to expansion levels in the MPR/FSEL LSTP, and may occur only if there is a near surface delamination (loss of rebar cover) over a large area due to structural deformation or distress.

The NRC staff notes that the potential rebar yield or slip issue under service conditions is not a concern for containment because the design code (ASME Code, Section III, Division 2) is based on the working stress design philosophy in which the maximum permissible stress in the rebar under service load (including ASR) combinations is half the yield stress.

The NRC staff reviewed the response to RAI-D11, Request 2, in the context of Category I structures (other than containment) designed to the ultimate strength philosophy, and found it acceptable because the revised MD uses in-plane expansion at ASR Severity Zone 4 level (CI or CCI exceeding 2 mm/m) to trigger one or more reasonable progressive actions (i.e., petrography, further analysis, retrofit) to evaluate potential rebar yield or slip (both local or over larger areas) due to ASR under service conditions. The NRC staff's concerns in RAI-D8 and RAI-D11 are resolved.

Based on its responses to RAI-D8 and RAI-D11, the NRC staff finds that the proposed method of evaluation provides reasonable assurance that strains in the reinforcement of ASR-affected structures remain within elastic limits under unfactored, normal operating (service) load conditions.

3.3.6 Description of Computer Programs Used for Finite Element Analysis

Enclosure 1, Section 3.4, "Summary of ASR and Structure Deformation Methodology Changes," of the letter dated August 1, 2016 (Reference 1), states that computer program ANSYS Mechanical ANSYS Parametric Design Language (APDL), Version 15.0, is used for the analytical and detailed evaluations of seismic Category I structures with deformation. The licensee noted that ANSYS has been used for design analyses of seismic Category I structures at other nuclear plants (e.g., Vogtle Electric Generating Plant, Units 3 and 4; and Virgil C. Summer Nuclear Station, Units 2 and 3). Section 6.2.1, "Analysis Models," in Enclosure 2 of the letter dated September 30, 2016 (Reference 2), further states that ANSYS Version 15 was procured as a nuclear quality assurance (QA) package, and has been validated and verified in accordance with SGH Quality Assurance Manual for Nuclear Facility Work (QANF) Program.

The NRC staff notes the ANSYS computer program has been previously used in analysis of nuclear safety-related structures, the computer program is recognized in the public domain, and has a sufficient history of being successfully used for structural analyses. Additionally, the NRC staff noted that the ANSYS Mechanical APDL Version 15.0 computer program used in this LAR was procured, validated and verified in accordance with the SGH nuclear QA program; therefore, the staff finds its use acceptable.

3.3.7 NRC Staff Conclusion on Proposed Evaluation Method for ASR-Affected Structures

Based on its review, the NRC staff finds that the licensee has proposed a reasonable method for analyzing structures affected by ASR. The proposed analysis methodology includes a reasonable approach for developing the load due to ASR (including the load from ASR in the concrete backfill), identifying acceptance criteria and expansion limits (i.e., threshold limits) to ensure impacted structures remain capable of performing their intended function, and maintaining stresses and strains within the elastic range under service loads. In addition, the licensee has developed reasonable load factors for the ASR load and has provided adequate justification for the proposed modifications or supplements to the existing codes of record. Therefore, the NRC staff finds the licensee's proposed analysis methodology acceptable for ASR-affected structures.

3.4 Monitoring of ASR Progression SMP

3.4.1 Monitoring for ASR Impact on Structural Limit States

Enclosure 1, Section 2.2 of the letter dated August 1, 2016 (Reference 1), notes that the licensee evaluated ASR material effects and concluded that no adjustments to structural properties are necessary for design evaluations when the extent of ASR is less than the limits from the MPR/FSEL LSTP. Section 3.5 discusses the SMP and notes that periodic visual inspections and expansion measurements will be used to monitor the progression of ASR expansion and building deformation.

Enclosure 1, Section 3.5.1 of the letter dated August 1, 2016 (Reference 1), discusses the monitoring approach for ASR expansion and notes that monitoring begins with monitoring of in-plane (x-y direction, or surface) expansion when visual indications of ASR are identified. In-plane expansion is measured using the CI or CCI. The CI is measured by overlaying a grid onto areas with ASR and measuring the crack widths that intersect the horizontal and vertical lines of the grid. The sum of crack widths is normalized by the length of the reference lines to determine the CI in-plane expansion, typically reported in mm/m. CCI is the weighted average of the CI in the two measured in-plane directions. Once in-plane expansion reaches 0.1 percent, extensometers are installed to measure through-wall (z direction) expansion thereafter. The expansion to-date is estimated using an empirical correlation developed during the MPR/FSEL LSTP (see SE Section 3.2.7 for further discussion of the correlation). The through-wall expansion is monitored and compared against limits developed based on the MPR/FSEL LSTP results. Reference 1 indicates that the SMP includes through-wall expansion limits for shear, flexure, and reinforcement anchorage, and in-plane limits for anchors.

Section 2.2 of the MD (Enclosure 3 of the letter dated June 7, 2018 (Reference 5)) notes that pin-to-pin distance measurements between two points on the concrete surface, using a calibrated mechanical device (capable of measuring length changes as small as 0.0001 inch), can also be used to determine in-plane expansion. However, these more precise measurements than CI are only capable of determining change in expansion after the pins have been installed because it provides change in length measurements between the pins at different times. Other measurements, such as CI or CCI, must be used to determine a "baseline" strain or expansion prior to installation of the pins. Total in-plane expansion can be determined by combining the baseline expansion up to installation of pins from CI measurements with change in expansion from pin-to-pin measurements.

The letter dated August 1, 2016 (Reference 1) also notes that the SMP includes the monitoring frequencies for areas impacted by ASR. Structures with signs of ASR are classified based on expansion to-date and higher levels of expansion are monitored more frequently. This information is summarized in Table 5 of Reference 1. Reference 1 notes that areas with visual indications of ASR are monitored on a 30-month interval and CCI monitoring begins when cracking can be accurately measured. These areas are referred to as "Tier 2." Once in-plane expansion reaches 0.1 percent, as measured by CCI, the area is classified as "Tier 3" and extensometers are installed, and the inspection interval is shortened to 6 months. Structures meeting the Tier 3 classification will also receive a structural evaluation to demonstrate their continued acceptability. This information is summarized and captured in Section 3.8.4.7.2 of the UFSAR markup.

The NRC staff reviewed the information in the letter dated August 1, 2016 (Reference 1), related to the proposed SMP in relation to monitoring methods and intervals. The acceptability of the

MPR/FSEL LSTP, and the resulting expansion limits, is discussed in Section 3.2 of this SE. During its review, the NRC staff noted that the expansion limits in Enclosure 1, Table 4 (Reference 1), do not match the limits identified in Enclosure 1, Table 2, or proposed Table 3.8-18 in the UFSAR markup. Additionally, it was unclear to the NRC staff how frequently monitoring of through-wall expansion would be conducted. To address this, the NRC staff issued RAI-M1 requesting the licensee to clarify the expansion limits and provide a justified interval for monitoring through-wall thickness. In its response to RAI-M1 by letter dated October 3, 2017 (Reference 3), the licensee stated that the through-thickness limit that will be used for monitoring is **[[ ]]**. The response also updated UFSAR Table 3.8-18, which references the limits in Report MPR-4288, with Footnote 2, which states, "the through-thickness expansion limit for shear, flexure and reinforcement anchorage presented in FP#101020 [MPR-4288] are different. The most limiting value is applied as the acceptance criterion for through-thickness expansion monitoring among these structural limit states." The response also added Footnote 3 to address volumetric expansion. Footnote 3 states, "the maximum observed maximum volumetric expansion for shear, flexure and reinforcement anchorage identified in FP#101050 [Report MPR-4273], Appendix B, Section 5 are different. The most limiting value is applied as the acceptance criterion for volumetric expansion monitoring among these structural limit states." These footnotes were included in the UFSAR markup to avoid a discussion of proprietary information in the UFSAR. The response also noted that through-thickness monitoring will be conducted on a 6-month interval.

The NRC staff reviewed the licensee's response to RAI-M1 and noted that the most limiting through-thickness value in Report MPR-4288, Section 2.1, is **[[ ]]**, which aligns with the value identified in the RAI response. The most limiting volumetric value in Report MPR-4273 is **[[ ]]**, which aligns with the information provided in response to RAI-M2, related to volumetric expansion. The NRC staff finds the licensee's response to RAI-M1 acceptable, because it clearly identifies the monitoring limits and clearly identifies a reasonable monitoring interval of 6 months for through-wall expansion (inspection interval adequacy is discussed further in the following paragraph).

The NRC staff noted that the SMP inspection frequency increases as ASR degradation progresses, moving from the standard SMP frequency (generally every 5 years for structures in environments likely to promote ASR) to every 6 months for Tier 3 structures. The NRC staff reviewed the inspection frequencies and finds them acceptable. Five years is an acceptable, conservative monitoring frequency for structures, as indicated in industry guidance documents, such as ACI 349.3R, "Evaluation of Existing Nuclear Safety-Related Concrete Structures" (Reference 32). Six months is a conservative inspection interval for structures, regardless of the degradation mechanism, and ASR is a slow-progressing degradation mechanism. Therefore, inspection frequencies that vary between 5 years and 6 months, depending on identified degradation, provide reasonable assurance that any future degradation will be identified and addressed before it could impact a structure's intended function.

The NRC staff also reviewed the proposed inspection or monitoring methods, which begin as visual inspections and progresses to CI/CCI (or CI/CCI supplemented by pin-to-pin expansion measurements) and through-wall expansion measurements as ASR degradation progresses. The NRC staff notes that visual inspection is the recommended, standard industry inspection method for routinely monitoring concrete structures to identify areas of potential structural distress or degradation, including degradation due to ASR. Once visual indications of ASR are identified, additional investigation is recommended. However, in this situation, the licensee has conservatively chosen to assume all visual indications of possible ASR are due to ASR. Once cracking is significant enough to reliably measure, a structure is identified as Tier 2 and monitored with CCI (or baseline CI/CCI supplemented by pin-to-pin expansion measurements). CCI provides a quantitative assessment of the extent of cracking and is a commonly used method for monitoring crack progression or in-plane expansion due to ASR, as discussed in ASR-monitoring specific guidance documents, such as U.S. Department of Transportation "Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures" (Reference 33). The NRC staff notes that the CI/CCI supplemented by pin-to-pin expansion measurements is a better monitoring approach (more accurate and more repeatable) for measuring in-plane ASR expansion, is also discussed in the Reference 33 guidance document, and is ideal for accurate threshold monitoring (SE Section 3.4.2). Once CCI (or in-plane expansion) values reach 1.0 mm/m, which is approximately 0.1 percent expansion, extensometers are installed and through-wall expansion is monitored. The transition to through-wall monitoring occurs at a conservative expansion value, which corresponds to a low level of ASR degradation as determined by the MPR/FSEL LSTP. At this point, 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, because it begins with industry-standard visual inspections, and moves to expansion monitoring as indications of ASR progress. Ultimately, volumetric expansion is monitored and compared to conservative limits determined during the MPR/FSEL LSTP. The progression of inspection methods (visual to CCI (or CI/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.

Based on its review, the NRC staff finds that the licensee's proposed SMP is acceptable to manage the impacts of ASR degradation on structural capacity because the program uses acceptable monitoring methods and intervals, which are paired with reasonable acceptance criteria, to ensure that expansion remains within the MPR/FSEL LSTP limits and, therefore, maintains design control of ASR-affected structures.

3.4.2 Threshold Monitoring for ASR Impact on Structural Analyses

Enclosure 1, Section 3.5 of the letter dated August 1, 2016 (Reference 1), discusses the SMP and notes that periodic visual inspections and expansion measurements will be used to monitor the progression of ASR expansion and building deformation. Section 3.5.2 includes the requirements for structures with measurable deformation. Structures are classified using the methodology described in Section 3.3 (discussed in Section 3.3 of this SE) and monitored in accordance with the intervals in Reference 1, Enclosure 1, Table 6. Stage 1 structures are monitored every 3 years, Stage 2 structures are monitored every 18 months, and Stage 3 structures are monitored every 6 months. Section 3.3.2 notes that each deformation evaluation results in a unique set of threshold monitoring parameters, along with associated acceptance criteria, or threshold limits. Section 6 of the MD provides additional guidance on how threshold monitoring parameters are chosen. The monitoring parameters should be quantifiable whenever possible and should be selected from Table 9 of the MD. Table 9 includes examples of possible parameters, including in-plane and through-thickness expansion, seismic isolation joints, individual crack widths/lengths, and deformation measurements. The MD also notes that qualitative measurements may be used to supplement quantitative measurements if necessary, but the purpose, type, and specific location of any qualitative measurement shall be clearly defined to enable reliable and repeatable data collection. Each parameter has a threshold limit associated with it that aligns with a fraction of the maximum allowable loads, including ASR loads. Section 7 of the MD notes that an additional 97 percent administrative limit is placed on the threshold limits, which are set at 90 percent, 95 percent, and 100 percent of the allowable, for Stage 1, 2, and 3, respectively. Section 7 also explains that if a threshold limit is approached, the structure can be reevaluated with a higher stage (more detailed) analysis or a structural modification can be performed to address the concern.

The NRC staff reviewed the information in the LAR and the MD related to the proposed SMP, as it relates to monitoring deformation. The staff noted that the SMP inspection interval begins with the standard interval (generally 5 years for structures in environments likely to promote ASR) and switches to 3 years once measurable deformation is identified. The interval decreases from 3 years to 6 months as a structure requires more detailed analyses and the ASR load moves closer to the code limit. Three years is an acceptable, conservative, monitoring interval for structures, and is less than the 5-year interval identified in industry guidance documents, such as ACI 349.3R. Eighteen months and six months are conservative inspection intervals for structures, regardless of the degradation mechanism. Varying the intervals for deformation monitoring between 3 years and 6 months, depending on how close a structure is to the allowable load limit, provides reasonable assurance that any future deformation will be identified and addressed before it impacts a structure's intended function.

The NRC staff also noted that each analysis results in unique threshold monitoring parameters and acceptance criteria, or threshold limit, for each structure. The MD provides guidance on the type of parameters that can be chosen and notes that when possible the parameters should be quantitative. The NRC staff reviewed the process, as described in the MD, and found it reasonable; however, much of the decision process relies on engineering judgement and may vary depending on the analysis. To verify that the licensee properly implemented the guidance and identified acceptable monitoring parameters, the NRC staff reviewed multiple calculations (covering all three analysis stages) and discussed the process with the licensee during a site audit the week of March 19, 2018. Based on the staff's discussion with the licensee, and its detailed review of the completed calculations and associated monitoring parameters, the NRC staff determined that the licensee was properly implementing the described methodology and was identifying reasonable monitoring parameters for each structure. The NRC staff also reviewed the approach for determining threshold limits and found it acceptable in Section 3.3.4 of this SE.

Based on its review, the NRC staff finds the licensee's proposed SMP acceptable to manage the impacts of ASR expansion and ASR loading on affected structures because the program uses acceptable monitoring parameters and intervals, which are paired with reasonable acceptance criteria, to ensure that ASR expansion remains within the limits in UFSAR markup Table 3.8-18 and that structures remain within the code allowable limits and, therefore, maintains design control of ASR-affected structures.

3.5 UFSAR Markup

Enclosure 1, Section 2.2 of the letter dated August 1, 2016 (Reference 1), provides a summary of the proposed changes or additions to the safety analysis in the Seabrook UFSAR. The proposed changes are necessary to incorporate into the licensing basis the proposed method of evaluation and associated monitoring program for ASR-affected seismic Category I concrete structures at Seabrook. The UFSAR markup pages are provided as Attachment 1 to Enclosure 1 of Reference 1, and have been amended during the NRC staff review by the submittals dated October 3, 2017; December 11, 2017; and June 7, 2018 (References 3, 4, and 5, respectively). The changes are in Sections 3.8.1, 3.8.3, 3.8.4, 3.8.6, and 3.9(B) of the UFSAR and define ASR loading as a design-basis load, provide a summary of how the ASR load is determined (including the load from expansion of concrete backfill), update related design and analysis procedures, and add associated monitoring programs. The markup also notes (Sections 3.8.4.4 and 3.9(B)) that the capacity of structural members and embedded concrete anchors in ASR-affected concrete is not reduced when ASR expansion levels are below the limits included in UFSAR markup Table 3.8-18 (as amended in Enclosure 2 of Reference 5), which was added in its entirety to capture ASR expansion limits. The tables identifying design load combinations and load factors for the containment and seismic Category I structures (Tables 3.8-1, 3.8-14, and 3.8-16) were updated to include the ASR load and associated load factors.

The NRC staff reviewed the information provided in the UFSAR markup and noted that the expansion limits in Reference 1, Enclosure 1, Table 4 do not match the limits identified in Enclosure 1 Table 2 or proposed Table 3.8-18 in the UFSAR markup, and that the monitoring interval for through-thickness expansion is not clear. To address this, the NRC staff issued RAI-M1 requesting the licensee to clearly identify the expansion limits and monitoring interval. In its response by letter dated October 3, 2017, the licensee provided a new UFSAR markup page which clearly identified the limits and the monitoring interval. The NRC staff reviewed the updated UFSAR markup and found it acceptable because it clarified the monitoring interval and the acceptance criteria for the monitoring program, a detailed evaluation of which is included in Section 3.4 of this SE.

The NRC staff also noted that the markup did not include any changes to UFSAR Section 3.8.5, "Foundations," to account for the effects of ASR. In addition, Section 3.3 of the letter dated August 1, 2016 (Reference 1), described how structural evaluations will be performed on structures impacted by ASR; however, no discussion was provided for how ASR in building foundations will be addressed. Since concrete foundations of Seabrook Category I structures use the same reactive aggregate as the superstructure, it was unclear whether foundations were evaluated for the impacts of ASR, and whether UFSAR Section 3.8.5 needed to be updated to account for ASR effects. Therefore, the NRC staff issued RAI-D1 requesting the licensee to explain how the concrete foundations of Seabrook Category I structures have been or will be evaluated for ASR.

In its response to RAI-D1 by letter dated October 3, 2017, the licensee stated that UFSAR Section 3.8.5, which provides the requirements for foundations, refers to other UFSAR sections for design requirements, including applicable codes, loading, acceptance criteria, and other requirements. These referenced sections, namely Section 3.8.1 for containment and Section 3.8.4 for Category I structures other than containment, have been revised to address structures with concrete affected by ASR. The licensee thus concluded that the UFSAR as marked up includes requirements for evaluating foundations affected by ASR; therefore, revision of UFSAR Section 3.8.5 is not necessary. The licensee further stated that the foundations of all Category I structures are evaluated or are being evaluated to meet the UFSAR Subsections 3.8.5.2 and 3.8.5.3 design requirements; these foundation evaluations will be included in the calculations summarizing the structural evaluation for each of the Category I structures as they are completed.

The NRC staff finds the licensee's response to RAI-D1 acceptable because it clarified that: (1) the UFSAR, as amended by the LAR, includes requirements for evaluating foundations affected by ASR by reference from UFSAR Section 3.8.5 to other specific UFSAR Sections (e.g., 3.8.1, 3.8.4) that include requirements for addressing ASR; and (2) evaluation of foundations of each ASR-affected Category 1 structure to meet the requirements for foundations in UFSAR Section 3.8.5 are, or will be, included in the structural evaluation calculations for ASR. The NRC staff's concerns in RAI-D1 are resolved.

During its review, the NRC staff noted that portions of the proposed actions related to the methodology for analyzing structures were not captured in the UFSAR. This included discussion of the future corroboration of MPR/FSEL LSTP specimens with Seabrook structures, and a description of the analysis methodology, specifically the determination of the ASR load and the supplements to the existing codes of record. Therefore, the staff issued RAI-D14 requesting the licensee to summarize future actions and the departures from the codes of record in the UFSAR.

In its response to RAI-D14 by letter dated June 7, 2018 (Reference 5), the licensee provided an updated UFSAR markup, including an update of Table 3.8-18, which included a summary description of the corroboration study and behavior assessment. The update also included revised discussions of the ASR load, which explained how the load is developed.

The NRC staff reviewed the licensee's response and the updated UFSAR markup. The NRC staff noted that updated Table 3.8-18 includes Footnote 4, which details the future expansion behavior assessments and the expansion curve corroboration study. The table also includes Footnote 5 which summarizes how pre-instrument expansion is determined and includes a reference to FP#100918 (Report MPR-4153, Revision 3 (Reference 34)), which provides the detailed method for determining pre-instrument expansion. The updated UFSAR also includes the list of "supplements" to the codes of record and a summary of how the ASR load is developed. The discussion of the ASR load development references FP#101196 (Revision 1 of the "Methodology Document"), which provides the detailed methodology for determining the ASR load.

The NRC staff finds the licensee's response to RAI-D14 acceptable because the updated UFSAR markup captures the future confirmatory actions (i.e., behavior assessments and expansion curve corroboration), the implementation of which will provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures. In addition, the discussion of the code supplements and the reference to the MD provide a description of the plant-specific aspects of the proposed analysis method.

Based on its review, including its review of RAIs M1, D1, and D14, and the license condition described in SE Section 3.6 below, the NRC staff finds that the proposed changes in the UFSAR markups are acceptable because they provide an adequate description of the proposed method of evaluation, including appropriate technical justification, for Seabrook ASR-affected seismic Category I structures, as required by 10 CFR 50.34(b).

3.6 License Condition

During its review of the MPR/FSEL LSTP, as described above in SE Section 3.2.8, the NRC staff determined that a license condition was necessary to capture the future confirmatory actions (i.e., behavior assessments and expansion curve corroboration) outlined in Footnote 4 of UFSAR markup Table 3.8-18. As described in this SE, the NRC staff concludes that the representative nature of the MPR/FSEL LSTP provides reasonable assurance that the results are currently bounding for Seabrook structures and that the expansion behavior is expected to be similar in the future. The large scale of the specimens, along with the reinforcement detailing and the concrete mix design, make the test results more representative of Seabrook structures than any existing literature data. Additionally, the results of the testing were consistent and repeatable across the specimens and the test methods aligned with the ACI test methods used to develop the Seabrook code of record design equations. Further, the ASR expansion levels achieved are greater than current levels on affected Seabrook structures.

However, the MPR/FSEL LSTP is unique since most existing ASR studies have reviewed the effects of ASR on small concrete specimens with little or no reinforcement. Other than the MPR/FSEL LSTP results and initial Seabrook expansion results, there is not a large body of information on the effects of ASR on in-situ structural performance. Additionally, the use of the test results in the proposed fashion is a first-of-a-kind application. Therefore, to ensure that the licensee continues to analyze additional, in-situ expansion data as it becomes available, and to ensure the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures, certain future confirmatory actions should be taken to verify that expansion behavior remains similar between the test specimens and Seabrook structures and that the test results continue to bound Seabrook.

To address this, the NRC staff developed the following license condition, based on the licensee's commitment in Footnote 4 of UFSAR markup Table 3.8-18, to ensure that appropriate verification of expansion behavior will be conducted in the future. The condition ensures that the licensee continues to gather and analyze expansion data of in-situ structures and ensures that the structure's expansion behavior continues to align with the expansion behavior seen in the MPR/FSEL LSTP specimens. The implications of any adverse findings from the confirmatory actions in the license condition will be appropriately addressed by the licensee in its Corrective Action Program in accordance with Item XVI, "Corrective Action," of 10 CFR Part 50, Appendix B, and is subject to NRC oversight as appropriate. This condition will be added to the table in Appendix C of the operating license.

License Condition:

The licensee will perform the following actions to confirm the continued applicability of the MPR/FSEL large-scale testing program conclusions to Seabrook structures (i.e., that future expansion behavior of ASR-affected concrete structures at Seabrook aligns with observations from the MPR/FSEL large-scale testing program and that the associated expansion limits remain applicable). The licensee shall notify the NRC each time an assessment or corroboration action is completed.

a. Conduct assessments of expansion behavior using the approach provided in Appendix B of Report MPR-4273, Revision 1 (Seabrook FP#101050), to confirm that future expansion behavior of ASR-affected structures at Seabrook Station is comparable to what was observed in the MPR/FSEL large-scale testing program and to check margin for future expansion. Seabrook completed the first expansion assessment in March 2018; and will complete subsequent expansion assessments every ten years thereafter.

b. Corroborate the concrete modulus-expansion correlation used to calculate pre-instrument through-thickness expansion, as discussed in Report MPR-4153, Revision 3 (Seabrook FP#100918). The corroboration will cover at least 20 percent of extensometer locations on ASR-affected structures and will use the approach provided in Appendix C of Report MPR-4273, Revision 1 (Seabrook FP#101050). Seabrook will complete the initial study no later than 2025 and a follow-up study 10 years thereafter.

3.7 <u>NRC Staff Technical Conclusion</u>

The NRC staff has reviewed the licensee's proposed method of evaluation for analyzing ASR-affected structures at Seabrook provided in the LAR, as well as conducted site audits and electronic audits, as documented above. Based on this review, the NRC staff concludes that the proposed plant-specific method of evaluation for design evaluation of seismic Category I reinforced concrete structures affected by ASR at Seabrook is acceptable and provides reasonable assurance that these structures continue to meet the relevant requirements of 10 CFR Part 50, Appendix A, GDC 1, 2, 4, 16 (containment only) and 50 (containment only), and 10 CFR Part 50, Appendix B. This conclusion is based on the following:

1. The licensee has met the requirements of GDC 1 by including ASR as a design-basis load and demonstrating that Seabrook ASR-affected structures will continue to meet the requirements of the codes of record (ACI 318-71 or ASME Section III, Division 2, 1975), as modified and supplemented in the LAR, for all design-basis loads and load combinations (including ASR) in the UFSAR. The licensee evaluated the codes of record to determine their applicability, adequacy, and sufficiency for reinforced concrete affected by ASR, by conducting research through the MPR/FSEL LSTP to study the effects of ASR on structural performance. The licensee developed the necessary supplements or modifications and limitations to the codes of record to demonstrate that structures continue to meet their intended functions. The MPR/FSEL LSTP was implemented in accordance with the quality assurance program requirements of 10 CFR Part 50, Appendix B, and was adequately developed to provide representative results for Seabrook structures.

2. The licensee has met the requirements of GDC 2 by including ASR as a design-basis load and demonstrating that Seabrook ASR-affected structures will continue to meet the requirements of the codes of record (ACI 318-71 or ASME Section III, Division 2, 1975), as modified and supplemented in the LAR, to incorporate ASR effects for all design-basis loads and load combinations (including ASR load) in the UFSAR, under normal and accident conditions along with the effects of environmental loadings such as earthquakes and other natural phenomena.

3. The licensee has met the requirements of GDC 4 by demonstrating that the ASR-affected structures will continue to meet GDC 1 and 2, as described above, because the design-basis loads and load combinations include the dynamic effects associated with missiles, pipe whipping, and discharging fluids, as applicable.

4. The licensee has met the requirements of GDC 16 and 50 by demonstrating that the containment will continue to meet GDC 1 and 2, as described above, for all design-basis loads and load combinations including ASR under normal and accident conditions.

5. The licensee has met the applicable requirements of 10 CFR Part 50, Appendix B, because the MPR/FSEL LSTP, which is a technical basis in support of the proposed method of evaluation, was implemented in accordance with the quality assurance program requirements of 10 CFR Part 50, Appendix B, and an SMP has been

established for monitoring the future progression of ASR expansion against the MPR/FSEL LSTP expansion limits and the structure-specific design output threshold monitoring limits up to which the design calculations remain valid.

6. The proposed method of evaluation is acceptable subject to the limitation that measured ASR expansion on affected Seabrook structures is within the limits of the MPR/FSEL LSTP as stated in UFSAR markup Table 3.8-18 and summarized in Table B below.

Table B: ASR Expansion Limits from MPR/FSEL LSTP

Structural Limit State	**ASR Expansion Limit**
Shear, Flexure, Reinforcement Anchorage	Through Thickness: **[[ ]]** Volumetric: **[[ ]]**
Anchors	In-plane: **[[ ]]**

Note: Compressive load from ASR in the direction of reinforcement is combined and evaluated with other applied loads.

7. The licensee's implementation of the future confirmatory actions required by the license condition discussed in Section 3.6 of this SE will provide assurance of the continued applicability of the MPR/FSEL LSTP conclusions to Seabrook structures.

4.0 STATE CONSULTATION

In accordance with the Commission's regulations, the New Hampshire State and Commonwealth of Massachusetts officials were notified of the proposed issuance of the amendment on December 21, 2018. The State officials had no comments.

5.0 ENVIRONMENTAL CONSIDERATION

The amendment changes requirements with respect to the installation or use of facility components located within the restricted area as defined in 10 CFR Part 20. The NRC staff has determined that the amendment involves no significant increase in the amounts, and no significant change in the types, of any effluents that may be released offsite, and that there is no significant increase in individual or cumulative occupational radiation exposure. The Commission has previously issued a proposed finding that the amendment involves no significant hazards consideration, and there has been no public comment on such finding published in the *Federal Register* on February 7, 2017 (82 FR 9604). Accordingly, the amendment meets the eligibility criteria for categorical exclusion set forth in 10 CFR 51.22(c)(9). Pursuant to 10 CFR 51.22(b), no environmental impact statement or environmental assessment need be prepared in connection with the issuance of the amendment.

6.0 FINAL NO SIGNIFICANT HAZARDS CONSIDERATION

The NRC staff's proposed no significant hazards consideration determination was published in the *Federal Register* on February 7, 2017 (82 FR 9604). On April 10, 2017, C-10 Research and Education Foundation, Inc. (C-10) filed a request for a hearing on this LAR (Agencywide Documents Access and Management System (ADAMS) Accession No. ML17100B013). On

October 6, 2017, the Atomic Safety and Licensing Board (Board) issued a Memorandum and Order granting C-10's hearing request (ADAMS Accession No. ML17279A968). On October 31, 2017, NextEra appealed the Board's decision (ADAMS Accession No. ML17304B075). On April 12, 2018, the Commission affirmed the Board's decision (ADAMS Accession No. ML18102A097).

Under the Atomic Energy Act of 1954, as amended, and the NRC's regulations, the NRC staff may issue and make an amendment immediately effective, notwithstanding the pendency before the Commission of a request for a hearing from any person, in advance of the holding and completion of any required hearing, where it has made a final determination that no significant hazards consideration is involved.

The NRC's regulation in 10 CFR 50.92(c) states that the NRC may make a final determination, under the procedures in 10 CFR 50.91, that a license amendment involves no significant hazards consideration if operation of the facility, in accordance with the amendment, would not: (1) involve a significant increase in the probability or consequences of an accident previously evaluated; or (2) create the possibility of a new or different kind of accident from any accident previously evaluated; or (3) involve a significant reduction in a margin of safety.

As required by 10 CFR 50.91(a), the licensee provided its analysis of the issue of no significant hazards consideration, which is presented below:

1. Does the proposed amendment involve a significant increase in the probability or consequences of an accident previously evaluated?

   Response: No.

   The proposed amendment is requesting approval of changes to the updated final safety analysis report (UFSAR) to allow a new method to analyze Alkali-Silica Reaction (ASR) related loads. The new methodology will verify that affected structures continue to have the capability to withstand all applied loads used in the original design of Seabrook structures. The proposed changes do not impact the physical function of plant structures, systems, or components (SSCs) or the manner in which SSCs perform their design function. The proposed changes do not alter or prevent the ability of operable SSCs to perform their intended function to mitigate the consequences of an event within assumed acceptance limits.

   The ASR-affected structures are not initiators of any accidents previously evaluated, and there are no accidents previously evaluated that involve a loss of structural integrity for seismic Category I structures. Approval of the UFSAR changes will demonstrate the structures affected by ASR will continue to maintain the capability to withstand all credible conditions of loading specified in the UFSAR.

   Therefore, the proposed changes do not involve a significant increase in the probability or consequences of an accident previously evaluated.

2. Does the proposed amendment create the possibility of a new or different kind of accident from any accident previously evaluated?

Response: No.

The proposed amendment is requesting approval of changes to the UFSAR to allow the use of a new method to analyze ASR related loads to verify that affected structures continue to have the capability to withstand applied loads used in the original design of Seabrook structures, with the addition of ASR loads and loads previously considered negligible. Approving the use of the new methodology will not create the possibility of a new or different kind of accident previously evaluated. The new methodology will demonstrate that structures continue to satisfy the design requirements of the code of construction and other applicable requirements with the additional load from ASR. Structures will respond to applied loads consistent with their original design.

The proposed changes to the UFSAR do not challenge the integrity or performance of any safety-related systems. The changes do not alter the design, physical configuration, or method of operation of any plant SSC. No physical changes are made to the plant, other than as a result of the revised monitoring program, so no new causal mechanisms are introduced.

Therefore, the proposed change does not create the possibility of a new or different kind of accident from any previously evaluated.

3. Does the proposed amendment involve a significant reduction in a margin of safety?

Response: No.

The proposed amendment is requesting approval of changes to the UFSAR to allow the use of a new method to analyze ASR related loads to verify that affected structures continue to have the capability to withstand all applied loads used in the original design of Seabrook structures.

The proposed methods for re-evaluating seismic Category I structures will demonstrate that structures satisfy the acceptance criteria in the current licensing basis when the loads associated with ASR expansion are included with other design loads and load combinations. The safety margin provided by the design codes in the current licensing basis will not be reduced since the proposed change is not requesting any change to the codes of record.

The proposed changes to the UFSAR do not affect the margin of safety associated with confidence in the ability of the fission product barriers (i.e., fuel cladding, reactor coolant system pressure boundary, and containment structure) to limit the level of radiation dose to the public. The proposed changes do not alter any safety analyses assumptions, safety limits, limiting safety system settings, or methods of operating the plant. The changes do not adversely impact plant operating margins or the reliability of equipment credited in the safety analyses. The proposed changes do not adversely affect systems that respond to safely shutdown the plant and to maintain the plant in a safe shutdown condition.

Therefore, the proposed change does not involve a significant reduction in a margin of safety.

The NRC staff reviewed the licensee's no significant hazards consideration determination. Based on this review and on the staff's evaluation of the underlying LAR as discussed above, the NRC staff concludes that the three standards of 10 CFR 50.92(c) are satisfied. Therefore, the NRC staff has made a final determination that no significant hazards consideration is involved for the proposed amendment and that the amendment should be issued as allowed by the criteria contained in 10 CFR 50.91.

7.0 CONCLUSION

The Commission has concluded, based on the considerations discussed above, that: (1) there is reasonable assurance that the health and safety of the public will not be endangered by operation in the proposed manner, (2) there is reasonable assurance that such activities will be conducted in compliance with the Commission's regulations, and (3) the issuance of the amendment will not be inimical to the common defense and security or to the health and safety of the public.

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SUBJECT: SEABROOK STATION, UNIT NO. 1 – ISSUANCE OF AMENDMENT NO. 159 RE: METHODOLOGY FOR ANALYSIS OF SEISMIC CATEGORY I STRUCTURES WITH CONCRETE AFFECTED BY ALKALI-SILICA REACTION (CAC NO. MF8260; EPID L-2016-LLA-0007) DATED MARCH 11, 2019

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