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

September 28, 2018

MEMORANDUM TO: Andrea D. Veil, Executive Director
Advisory Committee on Reactor Safeguards

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

SUBJECT: SEABROOK STATION, UNIT NO. 1 – SUBMISSION OF ALKALI-SILICA REACTION LICENSE AMENDMENT REQUEST DRAFT SAFETY EVALUATION TO SUPPORT THE ADVISORY COMMITTEE ON REACTOR SAFEGUARDS' REVIEW OF SEABROOK LICENSE RENEWAL (CAC NO. MF8260; EPID L-2016-LLA-0007)

Enclosed is the Office of Nuclear Reactor Regulation (NRR) staff's draft safety evaluation regarding its review of the Seabrook Station, Unit No. 1, license amendment request (LAR) related to alkali-silica reaction. By letter dated August 1, 2016 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML16216A250), as supplemented by letters dated September 30, 2016; October 3, 2017; October 17, 2017; December 11, 2017; and June 7, 2018 (ADAMS Accession Nos. ML16279A047, ML17277A337, ML17291B136, ML17345A641, and ML18158A540, respectively), NextEra Energy Seabrook, LLC submitted LAR No. 16-03, requesting changes to the Updated Final Safety Analysis Report to include methods for analyzing seismic Category I structures with concrete affected by alkali-silica reaction.

This draft safety evaluation is being provided to support the Advisory Committee on Reactor Safeguards' meeting of the Subcommittee on Plant License Renewal scheduled for October 31, 2018, in Rockville, Maryland. This draft safety evaluation is being provided as supporting information on NRR's efforts related to closing out the license renewal open item on alkali-silica reaction.

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Enclosure 2 to this memorandum contains proprietary information.
When separated from Enclosure 2, this document is DECONTROLLED.

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The NRR staff has determined that the draft 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 NRR staff has also prepared a non-proprietary publicly available version of the safety evaluation, which is provided as Enclosure 1. The proprietary version of the safety evaluation is provided as Enclosure 2.

Docket No. 50-443

Enclosures:
1. Draft Safety Evaluation (non-proprietary)
2. Draft Safety Evaluation (proprietary)

cc w/o Enclosure 2: Listserv

Enclosure 1

NON-PROPRIETARY DRAFT SAFETY EVALUATION

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
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WASHINGTON, D.C. 20555-0001

DRAFT SAFETY EVALUATION BY THE OFFICE OF NUCLEAR REACTOR REGULATION

RELATED TO AMENDMENT NO. XXX 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, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Revision 3, dated October 2012 (Reference 7) for analyses of ASR loads. During the course of the review, the licensee determined that the "100-40-40" procedure, as discussed in RG 1.92, was not necessary and 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.

Criterion 1 – Quality standards and records

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 <u>Shear Test Program - Results and Conclusions</u>

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

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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.

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3.2.4 <u>Reinforcement Anchorage Test Program - Results and Conclusions</u>

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 [illegible], "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 ASR Impacts on Other Limit States and Design Implications (Reference 1, Enclosure 5, Sections 5 and 6)

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.

<u>Reinforcement Strain</u>

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.

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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 structures' 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 Methodology for Determination of Through-Wall Expansion to Date at Seabrook

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 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 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

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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, 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 Development of ASR Loads, Including ASR Expansion in Concrete Backfill

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 Modifications or Supplements to the Codes of Record

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.

<u>Supplement 2</u> – 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 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 structures' 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 NRC Staff Technical Conclusion

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: [[ ]]

<u>Note</u>: 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.

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SUBJECT: SEABROOK STATION, UNIT NO. 1 – SUBMISSION OF ALKALI-SILICA REACTION LICENSE AMENDMENT REQUEST DRAFT SAFETY EVALUATION TO SUPPORT THE ADVISORY COMMITTEE ON REACTOR SAFEGUARDS' REVIEW OF SEABROOK LICENSE RENEWAL (CAC NO. MF8260; EPID L-2016-LLA-0007) DATED SEPTEMBER 28, 2018

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