May 18, 2018

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Docket No. 50-443
SBK-L-18072

U. S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, DC 20555-0001

Seabrook Station
Revised Structures Monitoring Aging Management Program

References:

1. NextEra Energy Seabrook LLC, letter SBK-L-10077, "Seabrook Station Application for Renewed Operating License," May 25, 2010 (Accession Number ML101590099).

2. NextEra Energy Seabrook LLC, letter SBK-L-17180, "Supplement 58 - Revised Alkali-Silica Reaction Aging Management Program," November 3, 2017 (Accession Number ML ML17310B540).

3. NextEra Energy Seabrook LLC, letter SBK-L-17155, "Supplement 58 - Response to Request for Additional Information for the Review of the Seabrook Station License Renewal Application – Building Deformation Analyses Related to Concrete Alkali-Silica Reaction," October 3, 2017 (Accession Number ML17277B519).

In Reference 1, NextEra Energy Seabrook, LLC (NextEra Energy Seabrook) submitted an application for a renewed facility operating license for Seabrook Station Unit 1 in accordance with the Code of Federal Regulations, Title 10, Parts 50, 51, and 54.

In Reference 2, NextEra Energy Seabrook submitted letter SBK-L-17180, providing an Updated Final Safety Analysis Report Sections A.2.1.31 for Structures Monitoring, A.2.1.31A for Alkali-Silica Reaction and A.2.1.31B for Building Deformation and revised LRA Appendix B Sections B.2.1.31 for Structures Monitoring, B.2.1.31A for Alkali-Silica

Reaction (ASR), and B.2.1.31B for Building Deformation Aging Management Programs.

During discussions with the staff during recent site audits several areas of discontinuity between the current Structures Monitoring Program and the submitted aging management program where identified. Enclosure 1 provides revised License Renewal Application (LRA) Appendix A - Updated Final Safety Analysis Report Sections A.2.1.31 for Structures Monitoring, A.2.1.31A for Alkali-Silica Reaction and A.2.1.31B for Building Deformation. Enclosure 2 provides revised LRA Appendix B Sections B.2.1.31 for Structures Monitoring, B.2.1.31A for Alkali-Silica Reaction (ASR) and B.2.1.31B for Building Deformation Aging Management Programs.

In Reference 3, NextEra Energy Seabrook submitted letter SBK-L-17155 responding to the Request for Additional Information (RAI). Based on questions from the staff Enclosure 3 provides clarification to the response previously provided in SBK-L-17155; Enclosure 1; Appendix A.

This letter contains no new or revised Commitments.

Enclosures 4-7 contain reports supporting the aging management program that have been revised since submittal to the docket. Enclosure 6 and 7 to this letter contains information proprietary to NextEra Energy Seabrook. This letter is supported by an affidavit (Enclosure 8), setting forth the basis on which the information in Enclosures 6 and 7 may be withheld from public disclosure by the Commission and addressing the considerations listed in 10 CFR 2.390(b)(4). Accordingly, it is respectfully requested that the information which is proprietary be withheld from public disclosure in accordance with 10 CFR 2.390. A non-proprietary version of these enclosures is provided in Enclosures 4 and 5.

If there are any questions or additional information is needed, please contact Mr. Edward J. Carley, Engineering Supervisor - License Renewal, at (603) 773-7957.

If you have any questions regarding this correspondence, please contact Mr. Kenneth Browne, Licensing Manager, at (603) 773-7932.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on May 18, 2018.

Sincerely,

Eric McCartney

Eric McCartney
Regional Vice President – Northern Region
NextEra Energy Seabrook, LLC

Enclosures:

Enclosure 1	Revised Seabrook Station License Renewal Application Updated Final Safety Analysis Report Sections A.2.1.31 for Structures Monitoring, A.2.1.31A for Alkali-Silica Reaction and A.2.1.31B for Building Deformation.
Enclosure 2	Revised Seabrook Station License Renewal Application Appendix B Sections B.2.1.31 for Structures Monitoring, B.2.1.31A for Alkali-Silica Reaction (ASR) and B.2.1.31B for Building Deformation Aging Management Programs.
Enclosure 3	Seabrook Station Clarifications to SBK-L-17155 Enclosure 1, Appendix A
Enclosure 4	MPR-4153, Revision 3, "Seabrook Station – Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction," July 2016 (Seabrook FP# 100918); (Non-proprietary)
Enclosure 5	MPR-4273, Revision 1, "Seabrook Station – Implication of large-Scale Test Program Results on Reinforced Concrete Affected by ASR," March 2018 (Seabrook FP# 101050); (Non-proprietary)
Enclosure 6	MPR-4153, Revision 3, "Seabrook Station – Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction," July 2016 (Seabrook FP# 100918); (Proprietary)
Enclosure 7	MPR-4273, Revision 1, "Seabrook Station – Implication of large-Scale Test Program Results on Reinforced Concrete Affected by ASR," March 2018 (Seabrook FP# 101050); (Proprietary)
Enclosure 8	NextEra Energy Seabrook, Application for Withholding Proprietary Information from Public Disclosure and Affidavit

cc:	D. H. Dorman	NRC Region I Administrator
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Enclosure 1 to SBK-L- 18072

Revised Seabrook Station License Renewal Application Updated Final Safety Analysis Report Section A.2.1.31 for Structures Monitoring, Section A.2.1.31A for Alkali-Silica Reaction (ASR) and Section A.2.1.31B for Building Deformation

A.2.1.31 STRUCTURES MONITORING PROGRAM

The Structures Monitoring Program includes the Masonry Wall Program and the Inspection of Water Control Structures Associated with Nuclear Power Plants Program.

The Structures Monitoring Program is implemented through the plant Maintenance Rule Program, which is based on the guidance provided in NRC Regulatory Guide 1.160 "*Monitoring the Effectiveness of Maintenance at Nuclear power Plants*" and NUMARC 93-01 "*Industry Guideline for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants*", and with guidance from ACI 349.3R, "*Evaluation of Existing Nuclear Safety-Related Concrete Structures*". The Structures Monitoring Program was developed using the guidance of these three documents. The Program is implemented to monitor the condition of structures and structural components within the scope of the Maintenance Rule, such that there is no loss of structure or structural component intended function.

A.2.1.31A ALKALI-SILICA REACTION (ASR) MONITORING

The plant specific ASR Aging Management Program manages cracking due to expansion and reaction with aggregates of concrete structures within the scope of License Renewal. The potential impact of ASR on the structural strength and anchorage capacity of concrete is a consequence of strains resulting from the expansive gel.

The Structures Monitoring Program and Section XI Subsection IWL Program perform visual inspections of the concrete structures at Seabrook Station for indications of the presence of alkali-silica reaction (ASR). ASR involves the formation of an alkali-silica gel which expands when it absorbs water. This expansion is volumetric in nature but is most readily detected by visual observation of cracking on the surface of the concrete. This cracking is the result of expansion that is occurring in the in-plane directions. Expansion is also occurring perpendicular (through the thickness of the wall) to the surface of the wall, but cracking will not be visible in this direction from the accessible surface. Cracking on the surface of the concrete is typically accompanied by the presence of moisture and efflorescence. Concrete affected by expansive ASR is typically characterized by a network or "pattern" of cracks. Micro-cracking due to ASR is generated through forces applied by the expanding aggregate particles and/or swelling of the alkali-silica gel within and around the boundaries of reacting aggregate particles. The ASR gel may exude from the crack forming white secondary deposits at the concrete surface. The gel also often causes a dark discoloration of the cement paste surrounding the crack at the concrete surface. If "pattern" or "map" cracking typical of concrete affected by ASR is identified, an evaluation will be performed to determine further actions.

ASR is primarily detected by non-intrusive visual observation of cracking on the surface of the concrete. The cracking is typically accompanied by the presence of moisture and efflorescence. ASR may also be detected or confirmed by removal of concrete cores and subsequent petrographic analysis.

Monitoring of crack growth is used to assess the in-plane expansion associated with ASR and to specify monitoring intervals. A Combined Cracking Index (CCI) is established at thresholds at which structural evaluation is necessary (see table below). The Cracking Index (CI) is the summation of the crack widths on the horizontal or vertical sides of 20-inch by 30-inch grid on the ASR-affected concrete surface. The horizontal and vertical Cracking Indices are averaged to obtain a Combined Cracking Index (CCI) for each area of interest. A CCI of less than the 1.0 mm/m (in-plane expansion of less than 0.1%) can be deemed acceptable with deficiencies (Tier 2). Deficiencies determined to be acceptable with further review are trended for evidence of further degradation. The change from qualitative monitoring to quantitative monitoring occurs when the Cracking Index (CI) of the pattern cracking equals or is greater than 0.5 mm/m (in-plane expansion of 0.05%) in the vertical and horizontal directions. Concrete crack widths less than 0.05 mm cannot be accurately measured and reliably repeated with standard, visual inspection equipment. A CCI of 1.0 mm/m (in-plane expansion of 0.1%) or greater requires structural evaluation (Tier 3). All locations meeting Tier 3 criteria will be monitored for in-plane expansion (via CCI or embedded pins), through-thickness expansion (via borehole extensometers), and volumetric expansion (using CCI or embedded pins and extensometer measurements) on a ½ year (6-month) inspection frequency. All locations meeting the Tier 2 structures monitoring criteria will be monitored on a 2.5 year (30-month) frequency. CCI correlates well with strain in the in-plane directions and the ability to visually detect cracking in exposed surfaces making it an effective initial detection parameter. In the event ASR monitoring results indicate a need to amend either the monitoring program acceptance criteria or the frequency of monitoring; NextEra Energy Seabrook will take such action under the Operating Experience element of the Alkali-Silica Reaction Aging Management Program.

Tier	Structures Monitoring Program Category	Recommendation for Individual Concrete Components	Criteria
3	Unacceptable (requires further evaluation)	• Structural Evaluation • Implement enhanced ASR monitoring, such as through-wall expansion monitoring using Extensometers.	1.0 mm/m (0.1%) or greater strain measurement (CCI or pin-pin)
2	Acceptable with Deficiencies	Quantitative Monitoring and Trending	• 0.5 mm/m (0.05%) or greater strain measurement (CCI or pin-pin) • CI or pin-pin measurement of greater than 0.5 mm/m (0.05%) in the vertical and horizontal directions
		Qualitative Monitoring	Any area with visual presence of ASR (as defined in FHWA-HIF-12-022) accompanied by a CI of less than 0.5 mm/m (0.05%) in the vertical and horizontal directions.
1	Acceptable	Routine inspection as prescribed by the Structural Monitoring Program	Area has no indications of pattern cracking or water ingress - No visual symptoms of ASR

The Alkali-Silica Aging Management Program was initially based on published studies describing screening methods to determine when structural evaluations of ASR affected concrete are appropriate. Large-scale destructive testing of concrete beams with accelerated ASR has confirmed that parameters being monitored are appropriate to manage the effects of ASR and that an acceptance criterion of 1 mm/m provides sufficient margin with regard to the effect of ASR expansion on structural capacity.

For heavily reinforced structures, in-plane expansion is limited. In-plane expansion measurements (i.e., CCI and embedded pin measurements) were observed in the large-scale test programs to plateau at a relatively low level of accumulated strain. While in-plane expansion monitoring (i.e., CCI and embedded pins) remains useful for the detection and monitoring of ASR at the initial stages, an additional monitoring parameter in the out-of-plane direction is required to monitor more advanced ASR progression. ASR expansion in the out-of-plane direction will be monitored by borehole extensometers installed in drilled core bore holes. In the selected locations, cores have and will continue to be removed for modulus testing to establish the level of through-thickness expansion to date. Instruments (extensometers) have and will continue to be placed in the resulting bore holes to monitor expansion in this direction going forward. The measured in-plane expansion and through-thickness expansion are used to determine volumetric expansion. Expansion measurements are used to maintain the limits specified below.

Structural Design Issue	**Criteria[1]**
Flexure & reinforcement anchorage	See FP#101020 - Section 2.1 for limit on through-thickness expansion See FP#101050 – Appendix B for limit on volumetric expansion
Shear	
Anchor bolts and structural attachments	See FP#101020 - Section 2.1 for limit on in-plane expansion

NextEra Energy Seabrook has and will continue to perform several actions to confirm that expansion behavior at the plant is consistent with the specimens from the large-scale test programs. These actions, described in the table below, assess similarity of expansion behavior in terms of trends between directions and expansion levels. These actions also include corroborating the correlation of normalized modulus versus through-thickness expansion derived from the large-scale test programs against plant data.

[1] Expansion Limit Criteria is considered proprietary to NextEra Energy Seabrook. FP #101020 MPR-4288, Revision 0, "Seabrook Station: Impact of Alkali-Silica Reaction on the Structural Design Evaluations," July 2016; FP#101050 MPR-4273, Revision 1, "Seabrook Station – Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction," March 2018; License Amendment Request 16-03, "Revise Current Licensing Basis to Adopt a Methodology for the Analysis of Seismic Category I; Structures with Concrete Affected by Alkali-Silica Reaction," August 1, 2016.

Periodic Confirmation of Expansion Behavior		**When**
Lack of mid-plane crack	Review of records for cores removed to date or since last assessment	Periodic assessments: At least 5 years prior to the Period of Extended Operations (PEO) Every 10 years thereafter
Expansion initially similar in all directions but becomes preferential in z-direction	Compare measured in-plane expansion (εxy) to through-thickness expansion (εz) using a plot of εz versus Combined Cracking Index (CCI)	
Expansions within range observed in test programs	Compare measured εxy, εz and εv (volumetric expansion) at the plant to limits from test programs to check margin for future expansion	
Corroborate modulus-expansion correlation with plant data (A secondary objective of these studies is to provide additional data to confirm that expansion behavior at the plant is comparable to the test specimens.)	For 20% of the 3 extensometer locations: • Remove cores for modulus Compare $\Delta\varepsilon$z determined from the modulus-expansion correlation with $\Delta\varepsilon$z determined from the extensometer and the original modulus result	At least 5 years prior to PEO (initial study) and 10 years thereafter (follow-up study). A detailed explanation of this approach is provided in MPR-4273, Revision 1, "Seabrook Station - Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction" (Seabrook FP# 101050).

A.2.1.31B BUILDING DEFORMATION MONITORING

The Building Deformation Aging Management Program is a plant specific program implemented under the existing Maintenance Rule Structures Monitoring Program. Building Deformation is an aging mechanism that may occur as a result of other aging effects of concrete. Building Deformation at Seabrook Station is primarily a result of ASR but can also result from swelling, creep, and shrinkage. Building deformation can cause components within the structures to move such that their intended functions may be impacted.

The Building Deformation Aging Management Program uses visual inspections associated with the Structures Monitoring Program and cracking measurements associated with the Alkali-Silica Reaction program to identify buildings that are experiencing deformation. The first inspection is a baseline to identify areas that are exhibiting surface cracking. The surface cracking is characterized and analytically documented. The first inspection identifies any local areas that are exhibiting deformation. The extent of surface cracking serves as input into an analytical model. This model determines the extent of building deformation and the frequency of required visual inspections.

For building deformation, location-specific measurements (e.g. via laser target and gap measurements) are compared against location-specific criteria to evaluate acceptability of the condition.
Structural evaluations are performed on buildings and components affected by deformation as necessary to ensure that the structural function is maintained. Evaluations of structures validate structural performance against the design basis, and use results from the large-scale test programs, as appropriate.

Evaluations for structural deformation consider the impact to functionality of affected systems and components (e.g. conduit expansion joints). NextEra Energy Seabrook will evaluate the specific circumstances against the design basis of the affected system or component. Structural evaluations will be used to determine whether additional corrective actions (e.g., repairs, additional inspections and/or analysis) to the concrete or components are required. Specific criteria for selecting effective corrective actions will be evaluated on a location-specific basis.

Enclosure 2 to SBK-L- 18072

Revised Seabrook Station License Renewal Application Section B.2.1.31 for Structures Monitoring, Section B.2.1.31A for Alkali-Silica Reaction (ASR) and Section B.2.1.31B for Building Deformation Aging Management Programs

B.2.1.31 STRUCTURES MONITORING PROGRAM

Program Description

The Seabrook Station Structures Monitoring Program (SMP) is an existing program that will be enhanced to ensure provision of aging management for structures and structural components including bolting within the scope of this program. The Structures Monitoring Program is implemented through the Seabrook Station Maintenance Rule Program, which is based on the guidance provided in NRC Regulatory Guide 1.160, Revision 2, *"Monitoring the Effectiveness of Maintenance at Nuclear Power Plants"* and NUMARC 93-01, Revision 2," *Industry Guidance for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants*", and with guidance from ACI 349.3R, "Evaluation of Existing Nuclear Safety-Related Concrete Structures". The Seabrook Station Structures Monitoring Program was developed using the guidance of these three documents to monitor the condition of structures and structural components within the scope of the Maintenance Rule, such that there is no loss of structure or structural component intended function.

The Seabrook Station Structures Monitoring Program includes periodic visual inspection of structures and structural components for the detection of aging effects specific for that structure. These inspections are completed by qualified individuals at a frequency determined by the characteristics of the environment in which the structure is found. A structure found in a harsh environment is defined as one that is in an area that is subject to outside ambient conditions, very high temperature, high moisture or humidity, frequent large cycling of temperatures, frequent exposure to caustic materials, or extremely high radiation levels. For structures in these harsh environments, the inspection is conducted on a five year basis (plus or minus one year due to outage schedule and two inspections within ten years). Structures not found in areas qualifying as a harsh environment are classified as being in a mild environment, and are inspected on a ten year basis (plus or minus one year due to outage schedule and two inspections within twenty years).

Individuals conducting the inspection and reviewing the results are qualified per the Seabrook Station Structures Monitoring Program, which is in accordance with the requirements specified in ACI 349.3R-96, *"Evaluation of Existing Nuclear Safety related Concrete Structures"*. Individuals conducting the inspection and reviewing the results are to possess expertise in the design and inspection of steel, concrete and masonry structures. These individuals must either be a licensed Professional Engineer experienced in this area, or will work under the direction of a licensed Professional Engineer experienced in this area.

The station SMP identifies plant equipment impacted or potentially impacted by building deformation through baseline and periodic walkdowns of the structures. The as-found conditions of the items of interest are evaluated and recommendations for repair or periodic monitoring are established in accordance with the Corrective Action Program.

Detection of aggressive subsurface environments will be completed through the sampling of groundwater. This procedure monitors groundwater for chloride concentration, sulfate concentration and pH on a 5 year basis

The Structures Monitoring Program will include an external surface inspection of the aboveground steel tanks 1-FP-TK-35-A, 1-FP-TK-35-B, 1-FP-TK-36-A, 1-FP-TK-36-B, and 1-AB-TK-29. This inspection will inspect the paint or coating for cracking, flaking, or peeling.

Examination of inaccessible areas, such as buried concrete foundations, will be completed during inspections of opportunity or during focused inspections. An evaluation of these opportunistic or focused inspections for buried concrete will be performed under the Maintenance Rule Program every 5 years (if no opportunistic inspection was performed during a 5-year period, a focused 5 year inspection is required) to ensure that the condition of buried concrete foundations on site is characterized sufficiently to provide reasonable assurance that the foundations on site will perform their intended function through the period of extended operation. To date Seabrook Station has performed numerous opportunistic inspections of buried concrete structures to confirm the characterization of ASR affected structures (e.g. switchyard generator step up transformer pit inspections in 2014, and Unit 2 Circulating Water Vault in 2015). Additional inspections may be performed in the event that an opportunistic or focused inspection or visible portions of the concrete foundation reveal degradation and will be entered into the Corrective Action Program (CAP).

Concrete structures were constructed equivalent to recommendations in ACI 201.2R, *"Guide for Making a Condition Survey of Concrete in Service"*. Loss of material due to leaching of calcium hydroxide is considered to be an aging effect requiring management for Seabrook Station. There have been indications of leaching in below grade concrete in Seabrook Station structures. Leaching of calcium hydroxide from reinforced concrete becomes significant only if the concrete is exposed to flowing water. Resistance to leaching is enhanced by using a dense, well-cured concrete with low permeability. These structures are designed in accordance with ACI 318 and constructed in accordance with ACI 301 and ASTM standards. Nevertheless, Seabrook Station manages loss of material due to leaching of calcium hydroxide with visual inspection through the Structures Monitoring Program.
Seabrook Station has scheduled specific actions to determine the effects of aggressive chemical attack due to high chloride levels in the groundwater. Seabrook Station has scheduled concrete testing during the second and third quarter of 2010. An evaluation will be performed based on the results of the testing and a determination of the concrete condition which may lead to additional testing or increased inspection frequency. Testing of concrete may consist of the following:

a. concrete core samples
b. penetration resistance tests
c. petrographic analysis of the concrete core samples
d. visual inspection of rebar as they are exposed during the concrete coring

NextEra Energy Seabrook will evaluate the results of the testing and, if required, undertake additional corrective actions in accordance with the Structures Monitoring Program CAP.

The Seabrook Station Structures Monitoring Program does not credit protective coatings for management of aging effects on structures and structural components within the scope of this program.

There are no preventative actions specified in the Seabrook Station Structures Monitoring Program, which includes implementation of NUREG-1801 XI.S5, XI.S6, and XI.S7. These are monitoring programs only.

The parameters monitored in the Seabrook Station Structures Monitoring Program are in agreement with ACI 349.3R-96 and ASCE 11-90, "*Structural Condition Assessment of Buildings*".

Concrete deficiencies are classified using the criteria specified in the Seabrook Station Structures Monitoring Program, which is based on the guidance provided in ACI 201.1R-2, "*Guide for Making a Condition Survey of Concrete in Service*".

As noted in the Seabrook Station response to NRC IN 98-26, *"Settlement Monitoring and Inspection of Plant Structures Affected by Degradation of Porous Concrete Subfoundations"*, porous concrete was not used in the construction of building sub-foundations at Seabrook Station.

Monitoring of structures and structural components in the scope of the Seabrook Station Structures Monitoring Program is performed in compliance with Regulatory Position 1.5 of NRC Regulatory Guide 1.160. The condition of all structures within the scope of this program is assessed on a periodic basis as specified by 10 CFR 50.65. Structures that do not meet their design basis at the time of inspection due to the extent of degradation, or that may not meet their design basis at the next normally scheduled inspection due to further degradation without intervention are entered into the Corrective Action Program and evaluated for corrective action and/or additional inspections as delineated in 10 CFR 50.65(a)(1). In addition, structures may also be scheduled for follow-up inspections following the completion of any corrective actions to that structure.

The condition of any structure subject to additional inspections or corrective actions is recorded through Seabrook Station Structures Monitoring Program reports to provide a basis for scheduling additional inspections and any required corrective actions in the future, as specified the Seabrook Station Structures Monitoring Program.

Structures that are determined to be acceptable under the Maintenance Rule structural inspections are monitored as specified in 10 CFR 50.65(a)(2).

Evaluations of a structure's condition assess the extent of any degradation of the structural member in accordance with industry standards and the judgment of the qualified individuals performing the inspections.

The acceptance guidelines in the Seabrook Station Structures Monitoring Program are a three-tier hierarchy similar to that described in ACI 349.3R-96, which provides quantitative degradation limits. Under this system, structures are evaluated as being acceptable, acceptable with deficiencies, or unacceptable. Evaluations of a structure's condition are completed according to the guidelines set forth in the Seabrook Station Structures Monitoring Program.

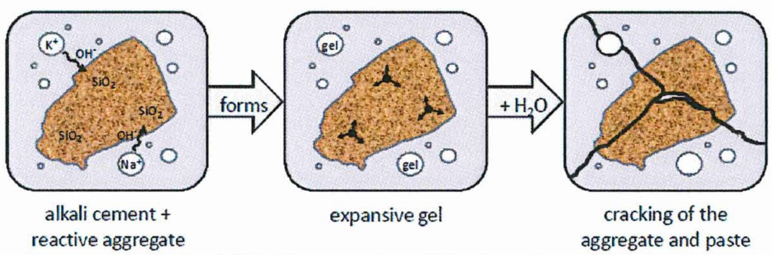
B.2.1.31A ALKALI-SILICA REACTION

PROGRAM DESCRIPTION

The Alkali-Silica Reaction (ASR) Aging Management Program (AMP) is a new plant specific program being implemented under the existing Maintenance Rule Structures Monitoring Program that will manage the aging effects related to Alkali-Silica Reaction of each structure and component subject to an Aging Management Review, so that the intended function(s) will be maintained consistent with the current licensing basis for the period of extended operation.

Alkali-Silica Reaction

Alkali-Silica Reaction is an aging mechanism that may occur in concrete under certain circumstances. It is a reaction between the alkaline cement and reactive forms of silicate material (if present) in the aggregate. The reaction, which requires moisture to proceed, produces an expansive gel material. This expansion results in strains in the material that can produce micro-cracking in the aggregate and in the cement paste. The potential impact of ASR on the structural strength and anchorage capacity of concrete is a consequence of strains resulting from the expansive gel. These strains produce the associated cracking. Because the ASR mechanism requires the presence of moisture in the concrete, ASR has been predominantly detected in groundwater impacted portions of below grade structures, with limited impact to exterior surfaces of above grade structures.



ASR Expansion Mechanism

Impact of Confinement

Reinforcing steel, loads on the concrete structure (i.e., deadweight of the structure itself), and the configuration of the structure provide confinement that restrains in-situ expansion of the gel and limits the resulting cracking in concrete.
Since the impact of ASR on mechanical properties relates to the extent of cracking, restraint of the expansion limits the reduction of in-situ mechanical properties and overall degradation of structural performance. There is a prestressing effect that occurs when reinforcement restrains the expansion caused by ASR. This effect is similar to concrete prestressing or analogous to pre-loading a bolted joint.

The concrete prestressing effect is only present when the concrete is confined. If the concrete is removed from the stress field, the concrete prestressing effect is lost. For example, a core taken from a reinforced concrete structure that has been affected by ASR will lose the confinement provided by the reinforcement and concrete surrounding the sample, and therefore is no longer representative of the concrete within its structural context.

Seabrook Station Concrete

The concrete mix designs used in original construction at Seabrook Station utilized an aggregate that was susceptible to ASR, which was not known at the time. Although testing was conducted in accordance with the ASTM C289 standards, the test method was subsequently identified as limited in its ability to predict long term ASR for moderate to low reactive aggregates. ASTM C289 has since been withdrawn.

In 2009, Seabrook Station tested seasonal groundwater samples to support the development of a License Renewal Application. The results showed that the groundwater had become aggressive and NextEra Energy Seabrook initiated a comprehensive review of possible effects to in-scope structures.

A qualitative walkdown of plant structures was performed and the "B" Electrical Tunnel was identified as showing the most severe indications of groundwater infiltration. Concrete core samples from this area were removed, tested for compressive strength and modulus of elasticity, and subjected to petrographic examinations. While the results showed that both compressive strength and modulus of elasticity had declined, the structure was determined to be within its design basis and therefore remained able to perform its design function. The results of petrographic examinations on the core samples identified Alkali-Silica Reaction (ASR). This discovery prompted an Extent of Condition evaluation. Because the ASR mechanism requires the presence of moisture or very high humidity in the concrete, ASR has been predominantly detected in portions of below-grade structures, with limited impact to exterior surfaces of above grade structures.

Large-Scale Testing Program

The structural assessment of ASR-affected structures at Seabrook Station considered the various limit states for reinforced concrete and applied available literature data to evaluate structural capacity. This evaluation identified gaps in the publicly available test data and the applicability to the reinforcement concrete at Seabrook Station. The limited available data for shear capacity and reinforcement anchorage for ASR-affected reinforced concrete with two-dimensional reinforcement mats were not representative of Seabrook Station. This conclusion was driven largely by the facts that the literature data for reinforcement anchorage were from a test method that ACI indicates is unrealistic and the literature data for shear capacity were from test specimens only inches in size. Additionally, no data were available on anchor bolt capacity on reinforced concrete with two dimensional reinforcement mats like Seabrook Station.

The need for Seabrook Station specific testing was driven by limitations 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. Although structural testing of ASR-affected test specimens has been performed, the application of the conclusions to a specific structure can be challenged by lack of representativeness in the data (e.g., small-scale specimens; poor test methods;

different reinforcement configuration). The large-scale test programs undertaken by NextEra Energy Seabrook provided data on the limit states that were essential for evaluating seismic Category I structures at Seabrook Station. The data produced from these programs were a significant improvement from the data in published literature sources, because test data across the range of ASR levels were obtained using a common methodology and identical test specimens. The results were used to assess the impact of ASR on structural limit states and on selected design considerations[2]. This assessment supports use of the test results in structural calculations.

The large-scale test programs included testing of specimens that reflected the characteristics of ASR-affected structures at Seabrook Station. Tests were completed at various levels of ASR cracking to assess the impact on selected limit states. The extent of ASR cracking in the test specimens was quantified by measuring the expansion in the in-plane and through-thickness dimensions. The in-plane dimension refers to measurements taken in a plane parallel to the underlying reinforcement bars. There was no reinforcement in the through-thickness direction (perpendicular to the in-plane direction). ASR expansion measurements were taken throughout the test programs. The test programs assessed flexural capacity and reinforcement anchorage, shear capacity, and capacity of anchor bolts and structural attachments to concrete~~)~~. The results of the shear and reinforcement anchorage test programs demonstrated that there was no adverse effect; on structural performance in these limit states when ASR expansion levels were below those in the test specimens. The results of the anchor test program demonstrated that there was no adverse effect on anchor capacity except at high levels of ASR expansion.

The effect of ASR on compressive strength was not assessed in the large-scale test program. An evaluation of compression using existing data from published literature sources was performed. The evaluation concluded that ASR expansion in reinforced concrete results in compressive load that should be combined with other loads in design calculations. However, ASR does not reduce the structural capacity of compression elements.

The specimens used in the large-scale test programs experienced levels of ASR that bound ASR levels currently found in Seabrook Station structures (i.e., are more severe than at Seabrook Station), but the number of available test specimens and nature of the testing prohibited testing out to ASR levels where there was a clear change in limit state capacity. Because there are no~~t~~ testing data for these more advanced levels of ASR, periodic monitoring of ASR at Seabrook is necessary to ensure that the level of ASR does not exceed that observed in the test programs, which ensures that the conclusions of the large-scale test program remain applicable.

The overall conclusion from analyses of structural limit states is that limit state capacity is not degraded when small amounts of ASR expansion are present in structures. Presently, the ASR expansion levels in Seabrook Station structures are below the levels at which limit state capacities are reduced.

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[2] FP #101020 MPR-4288, Revision 0, "Seabrook Station: Impact of Alkali-Silica Reaction on the Structural Design Evaluations," July 2016

One of the objectives of the large-scale test program was to identify effective methods for monitoring ASR. The program concluded that monitoring the in-plane and through-thickness expansion is effective for characterizing the significance of ASR in structures.

In-plane expansion can be monitored using embedded pins or the Combined Cracking Index (CCI). Embedded pin measurements determine changes in ASR expansion more precisely than CCI measurements over the duration of a monitoring period, since the embedded pin measurements are performed using a calibrated mechanical device capable of measuring changes in length as small as 0.0001 inch. However, embedded in-plane expansion measurements are only able to capture strains that occur after the gage points are installed in the concrete surface after initial (baseline) measurements are made. For use at Seabrook Station, CCI provides a reasonable value for expansion to date. Once the CCI is calculated and the expansion level is quantified, reference pin measurements can be used to monitor future expansion.

Snap ring borehole extensometers (SRBEs) provided accurate and reliable measurements for monitoring through-thickness expansion.

Results from the large-scale test program are used to support evaluations of structures subjected to deformation. These evaluations are discussed in the Building Deformation Aging Management Program in LRA Section B.2.1.31B.

PROGRAM ELEMENTS

The following provides the results of the evaluation of each program element against the 10 elements described in Appendix A of NUREG-1800 Rev. 1, *"Standard Review Plan for Review of License Renewal Applications for Nuclear Power Plants"*.

ELEMENT 1 - SCOPE OF PROGRAM

The Alkali-Silica Reaction (ASR) Aging Management Program (AMP) provides for management of aging effects due to the presence of ASR. The program scope includes concrete structures within the scope of the License Renewal Structures Monitoring Program and License Renewal ASME Section XI Subsection IWL Program. License Renewal concrete structures within the scope of this program include:

Category I Structures

- Containment Building (including equipment hatch missile shield)
- Containment Enclosure Building
- Containment Enclosure Ventilation Area
- Service Water Cooling Tower including Switchgear Rooms
- Control Building
- Control Building Make-up Air Intake Structures
- Diesel Generator Building
- Piping (RCA) Tunnels
- Main Steam and Feed Water East and West Pipe Chase
- Waste Processing Building

- Tank Farm
- Condensate Storage Tank Enclosure
- Emergency Feed Water Pump House Building, including Electrical Cable Tunnels and Penetration Areas (Control Building to Containment)
- Fuel Storage Building
- Primary Auxiliary Building including RHR Vaults
- Service Water Pump House
- Service Water Access (Inspection) Vault
- Circulating Water Pump House Building (below elevation 21'-0)
- Safety Related Electrical Manholes and Duct Banks
- Pre-Action Valve Building

Miscellaneous Non-Category I Yard Structures
- SBO Structure – Transformers and Switch Yard foundations
- Non-Safety-Related Electrical Cable Manhole, Duct Bank Yard Structures foundations
- Switchyard and 345 KV Power Transmission foundations

Non-Category I Structures
- Turbine Generator Building
- Fire Pump House
- Aboveground Exterior Tanks 1-FP-TK-35-A, 1-FP-TK-35-B, 1-FP-TK-36-A, 1-FP-TK-36-B and 1-FP-TK-29 foundations
- Fire Pump House Boiler Building
- Non-Essential Switchgear Building
- Steam Generator Blowdown Recovery Building
- Intake & Discharge Transition Structures

ELEMENT 2 - PREVENTIVE ACTIONS

There are no preventive actions specified in the Seabrook Station Structures Monitoring Program, which includes implementation of NUREG-1801 XI.S5, XI.S6, and XI.S7. These are monitoring programs only. Similarly, the ASR AMP does not rely on preventive actions.

ELEMENT 3 - PARAMETERS MONITORED/INSPECTED

The Alkali-Silica Reaction (ASR) AMP manages the effects of cracking due to expansion and reaction with aggregates. The potential impact of ASR on the structural performance and anchorage capacity of concrete is a consequence of strains resulting from the expansive gel. The strains consequently produce the associated cracking.

The program focuses on identifying evidence of ASR, which could lead to expansion due to the reaction with aggregates. The program reflects published guidance for condition assessment of structures and incorporates practices consistent with those used as part of the large-scale testing programs.

Initial screening of ASR

Walkdowns of the station are performed on a periodic basis (SMP walkdowns, Systems Walkdowns, etc.). Visual symptoms of deterioration are noted and compared to those commonly observed on structures affected by ASR. Common visual symptoms of ASR include, but are not limited to, "map" or "pattern" cracking and surface discoloration of the cement paste surrounding the cracks. The cracking is typically accompanied by the presence of moisture and efflorescence. The lists of symptoms associated with the initial screening of ASR is consistent with many published documents, including but not limited to the Federal Highway Administration (FHWA) document FHWA-HIF-09-004, "Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures", and the Institution of Structural Engineering document "Structural Effects of Alkali-Silica Reaction: Technical Guidance on the Appraisal of Existing Structures."

Inspection of inaccessible areas of concrete will be performed during opportunistic or focused inspections for buried concrete performed under the Maintenance Rule every 5 years. The concrete materials used to produce the concrete placed in inaccessible areas were the same as the concrete materials used to produce the concrete placed in accessible areas. Thus, the performance and aging of inaccessible concrete would be the same as the performance and aging of accessible concrete.

Since the concrete mix and aggregates used at Seabrook Station are consistent between structures, it is assumed unless demonstrated otherwise that pattern cracking observed during walkdowns is from ASR. Petrographic examination can be performed on a concrete specimen to aid in confirming the proposed diagnosis arrived upon from visual inspection of the concrete surface. Typical petrographic features of ASR generally consist of the following:

- Micro-cracking in the aggregates and/or cement paste
- Reaction rims around the aggregates.
- Silica gel filling cracks or voids in the sample.
- Loss of cement paste-aggregate bond.

Expansion

For ASR-affected surfaces at Seabrook Station, NextEra Energy Seabrook monitors the effects of ASR expansion by obtaining measurements in both the in-plane (X&Y directions) and through-thickness directions (Z-direction). Specifically, NextEra Energy Seabrook monitors the Combined Cracking Index (CCI) and/or embedded pin measurements (the distance between the embedded reference pins) for in-plane expansion and extensometer measurements for through-thickness expansion. In addition, NextEra Energy Seabrook uses the in-plane and through-thickness expansion measurements to determine volumetric expansion. Expansion from ASR results in cracking and a change to the material properties of the concrete, and eventually requires an evaluation to ensure adequate structural performance.

Expansion is a readily quantifiable parameter and an effective method for determining ASR progression. Expansion measurements at Seabrook Station can be easily obtained in the in-plane directions. The Cracking Index (CI) is a quantitative assessment of cracking present in the cover concrete of affected structures. A CI measurement is taken on accessible surfaces exhibiting the typical ASR symptoms. The CI is the summation of the crack widths on the horizontal or vertical sides of a section of the ASR-affected concrete surface of predefined dimensions. Seabrook Station uses a grid size of 20 inches by 30 inches. The CI in a given direction is converted and reported in units of mm/m. Embedded pins are also installed to measure strain and monitor in-plane expansion.

The CIs are used to establish the Combined Cracking Index (CCI). The CCI estimates expansion on a concrete surface using measurements of crack widths along a pre-determined length or grid. The CCI is calculated by summing the crack widths crossing all reference grid lines and dividing the result by the sum of all gridline lengths.

Criteria used in assessment of expansion is expressed in terms of in-plane expansion based on the screening approach described in MPR-3727, "Seabrook Station: Impact of Alkali-Silica Reaction on Concrete Structures and Attachments."

Initial screening for ASR is performed by using in-plane expansion measurements. In-plane strain values exceeding 1 mm/m (0.1%) will trigger additional actions. CCI is a relatively simple, non-destructive method for monitoring cracking that appropriately characterizes expansion until expansion reorients in the direction of least restraint (i.e., the through-thickness direction at Seabrook Station).

Results from the large-scale test programs indicated that direction of expansion is not significantly affected by the reinforcement when expansion is low. At higher levels of expansion, the two-dimensional reinforcement mats provide confinement in the in-plane directions, and through-thickness expansion dominates (MPR-4273, Revision 1).

Data analysis from the large-scale test program has been completed and thresholds have been established based on the test reports. The thresholds are based on the structure as a whole so if localized extensive ASR or macro cracking is experienced in particular areas of the structure, then the entire structure is assumed to be susceptible to similar degradation. The overall methodology for using in-plane expansion, through-thickness expansion, and volumetric expansion values for various aspects of the monitoring program is discussed below.

Anchor Performance Monitoring Parameter

For anchor performance, the large-scale test programs show that ASR does not have an effect until in-plane expansion reaches a sufficiently high level. Therefore, if the CCI exceeds a specified threshold, additional evaluation must be performed to justify continued acceptability of the anchors.

This approach is based on the fact that anchor performance is sensitive to in-plane expansion, but not through-thickness expansion. In-plane expansion creates micro-cracks parallel to the axis of an anchor, mainly in the concrete cover. These micro-cracks perpendicular to the concrete surface have the potential to provide a preferential failure path within a potential breakout cone, leading to degraded anchor performance.

Through-thickness expansion has the potential to create micro-cracks perpendicular to the axis of an anchor. These potential micro-cracks that open parallel to the concrete surface do not provide a preferential failure path to result in degraded anchor performance. An anchor loaded in tension would compress the through-thickness expansion and close any potential micro-cracks within the area of influence of that anchor. Without a 'short-circuit' of the breakout cone, through-thickness expansion is a non-factor in anchor performance.

Crack Width Summation

Crack width summation is a simple methodology for initial assessment of ASR-affected components and is recommended by publicly available resources.

ASR produces a gel that expands as it absorbs moisture. This expansion exerts a tensile stress on the surrounding concrete which strains the concrete and eventually results in cracking.

The engineering strain in a structural member at the time of crack initiation (ε_{cr}) is equivalent to the tensile strength of the concrete divided by the elastic modulus ($\varepsilon_{cr} = \sigma_t / E$). The Cracking Index quantifies the extent of the surface cracking. The total strain in the concrete can be approximated as the sum of the strain at crack initiation plus the cracking index ($\varepsilon \approx \varepsilon_{cr} + CI$). Figure 1 depicts a concrete specimen with rebar being put in tension resulting in cracking.
Concrete has little strain capacity; therefore, in ASR-affected concrete, the crack widths comprise most of the expansion (ΔL). As a result, the Cracking Index provides a reasonable approximation of the total strain applied to the concrete after crack initiation, because strain in the un-cracked concrete between cracks is minimal.

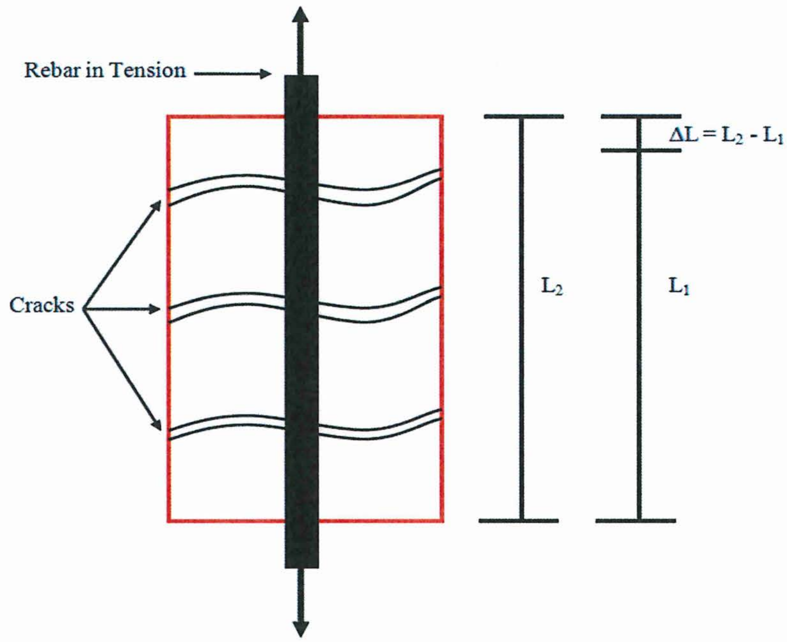

Figure 1 - Concrete Specimen put in Tension

For surfaces where horizontal and vertical cracking indices are similar (e.g., where there is equivalent reinforcement in both directions), a Combined Cracking Index (CCI) that averages the horizontal and vertical Cracking Indices can consolidate the expansion assessment to a single parameter.

Change in Elastic Modulus and Extensometer Measurements

The large-scale test program showed that through-thickness expansion dominates for structures with two-dimensional reinforcement mats in the in-plane directions (like structures at Seabrook Station).

Data from the structural testing programs showed that expansion in the in-plane direction plateaued at low expansion levels, while expansion in the through-thickness direction continues to increase. Based on this observation, Seabrook Station has and will continue to install the extensometers in Tier 3 and other selected locations to measure expansion in the through-thickness direction. This approach enables expansion to be measured for a given concrete structural member from the time the extensometer is installed and going forward. To calculate the total expansion, NextEra Energy Seabrook has and will determine expansion from original construction until the time the extensometers are installed and then add the extensometer measurement.

The method to determine the total ASR induced through-thickness expansion at each instrument location at Seabrook Station is to determine expansion at the time the extensometer is installed based on the reduction in modulus of elasticity.

The foundation of the approach for determining expansion in the through-thickness direction prior to installing an extensometer is the universal agreement among published sources that elastic modulus decreases with ASR progression. NextEra Energy Seabrook could have used the literature data to produce a generic correlation between reduction of elastic modulus and expansion, but instead elected to pursue a more precise relationship that was more representative of Seabrook Station. The correlation relating through-thickness expansion to elastic modulus is based exclusively on data from the large-scale test programs, which has several important advantages:

- All data are from cores removed from reinforced concrete that has a reinforcement configuration that is comparable to Seabrook Station. Accordingly, the test data reflect ASR development is a stress field that was more representative of an actual plant structure than literature data, which are typically based on unconfined cylinders.

- The cores were obtained from test specimens that have a concrete mixture design that is as representative of Seabrook Station as practical.

- The test programs were conducted under a Nuclear Quality Assurance program that satisfies the requirements of 10 CFR 50, Appendix B.

The extensometer measurements will provide direct measurements of through-thickness expansion going ~~forwards~~. The measurements are the parameter to be monitored. The elastic modulus will not be monitored going forward. Pre-instrument expansion is calculated initially to establish expansion to date and is not repeated (except for the purpose of studies to corroborate applicability of the correlation, which are discussed in the license renewal commitments).

Volumetric Expansion

To support that concrete at Seabrook Station is appropriately represented by the specimens from the large-scale test programs, NextEra Energy Seabrook will also monitor volumetric expansion using the CCI data and extensometer data.

Volumetric expansion is the sum of expansion in each of the principal directions, as shown in the equation below.

$$\varepsilon_v = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$$

Where:

ε_v = volumetric expansion
ε_1 = principal strain (e.g., in the length direction)
ε_2 = principal strain (e.g., in the height direction)
ε_3 = principal strain (e.g., in the depth direction)

Because Seabrook Station uses combined cracking index (CCI) to characterize in-plane expansion, this equation is re-written as follows:

$$\varepsilon_v = 2 \times (0.1 \times CCI) + \varepsilon_{TT}$$

Where:

ε_v = volumetric strain, %
CCI = combined cracking index, mm/m
ε_{TT} = through-thickness expansion, %

Structural Limit States

The applicable design codes provide methodologies to calculate structural capacities for the various limit states and loading conditions applicable to Seabrook Station. Each relevant limit state was evaluated using published literature and the results of the large-scale test programs that used specimens designed and fabricated to represent reinforced concrete at Seabrook Station. The following guidance applies for structural evaluations of ASR-affected concrete structures at Seabrook Station:

- Flexure/Reinforcement Anchorage - Based on the MPR/FSEL large-scale test program results, structural evaluations should consider that there has been no adverse impact on flexural capacity and reinforcement anchorage (development length) performance, provided that through-thickness expansion is at or below bounding conditions of the large-scale testing and expansion behavior is comparable to the test specimens, including through-thickness and volumetric expansion.

- Shear – Based on the MPR/FSEL large-scale test program results, structural evaluations should consider that there has been no adverse impact on shear capacity, provided that through-thickness expansion is at or below bounding conditions of the large-scale testing and expansion behavior is comparable to the test specimens, including through-thickness and volumetric expansion.

- Anchors and Embedments – Based on the MPR/FSEL large-scale test program results, structural evaluations should consider that there is no adverse effect to post-installed or cast in place anchor/embedment capacity, provided that in-plane expansions remain at or below limits established by large-scale testing. Through-thickness expansion is not relevant for anchor/embedment capacity.

The interim structural assessment used available literature to identify the impact of ASR on the various limit states. The literature review identified three limit states of interest: shear in members without transverse reinforcement, reinforcement anchorage in specimens without transverse reinforcement and embedments. For shear, the literature showed up to a 25% reduction in capacity but the results varied significantly within the literature. For reinforcement anchorage, the available test data were based on an unreliable test method. For embedments, there was no relevant data. The large-scale test programs addressed these limits using test specimens representative of reinforced concrete structures at Seabrook Station.

ELEMENT 4 - DETECTION OF AGING EFFECTS

Monitoring walkdowns are performed on a periodic basis. The Structures Monitoring Program (SMP) walkdowns identify areas that show symptoms of ASR being present. The SMP includes periodic visual inspection of structures and components for the detection of aging effects specific for that structure. The inspections are completed by qualified individuals at a frequency determined by the characteristics of the environment in which the structure is found. A structure found in a harsh environment is defined as one that is in an area that is subject to outside ambient conditions, very high temperature, high moisture or humidity, frequently large cycling of temperatures, frequent exposure to caustic materials, or extremely high radiation levels. For structures in these harsh environments, the inspection is conducted on a five (5) year basis (plus or minus one year due to outage schedule and two inspections within ten years. Structures not located in an area qualifying as a harsh environment are classified as being in a mild environment, and are inspected on a ten (10) year basis (plus or minus one year due to outage schedule and two inspections within twenty years).

In-Plane Expansion

As previously discussed in Element 3, Seabrook Station uses the CCI methodology or embedded pin measurements to monitor the expansion of ASR affected areas in the in-plane direction. An in-plane strain measurement of less than 1.0 mm/m (0.1%) can be deemed acceptable with deficiencies (Tier 2). Deficiencies determined to be acceptable with further review are trended for evidence of further degradation. An in-plane strain measurement of1.0 mm/m (0.1%) or greater requires structural evaluation (Tier 3). All locations meeting Tier 3 will be monitored on a ½ year (6-month) inspection frequency. All locations meeting Tier 2 will be monitored on a 2.5 year (30-month) frequency. In the event ASR monitoring results indicate a need to-amend either the monitoring program acceptance criteria or the frequency of monitoring, NextEra Energy Seabrook will take such action under the Operating Experience element of the Alkali-Silica Reaction Aging Management Program. (Structural calculations that support the Building Deformation AMP may indicate that more frequent CCI monitoring (e.g., semiannually) may be appropriate for locations that have an in-plane strain measurement of less than 1.0 mm/m (0.1%) (Tier 1 or 2). NextEra Energy Seabrook will perform in-plane expansion monitoring at whichever interval is more frequent.)

Seabrook Station has established reference grids that track the CCI of ASR affected areas. These grids are 20" x 30" and consist of three parallel vertical lines and two parallel horizontal lines. Measurement points (gage points) are installed at the intersections of horizontal and vertical lines of the reference grid to allow for long-term monitoring of potential ongoing expansion. The CI is obtained from measurements of crack widths along a set of lines drawn on the surface of a concrete member. Expansion is documented by measuring the increase in the length of the lines used to determine the CI (distance between gage points). A pocket-size crack comparator card and an optical comparator are used to take the measurements.

The location of the CCI reference grid is established in the area that appears to exhibit the most-severe deterioration due to ASR (accessibility and structure geometry also factor into the decision making progress on where to establish a grid). At Seabrook Station the axes of the reference grid/grids are parallel and perpendicular to the main reinforcement of the associated reinforced concrete member.

CI correlates well with strain in the in-plane directions and the ability to visually detect cracking in exposed surfaces making it an effective initial detection parameter. However, embedded pin measurements determine changes in ASR expansion more precisely than CI measurements over the duration of a monitoring period, since the embedded pin measurements are performed using a calibrated mechanical device. In addition, the embedded pin measurements avoid the potential increase due to human or environmental factors that do not relate to true expansion (i.e. the inadvertent widening of the face of a crack due to cleaning of the surface).

Embedded pin measurements are only able to capture strains that occur after the gage points are installed in the concrete surface after initial (baseline) measurements are made. For use at Seabrook Station, CI provides a reasonable value for expansion to date. Once the CI is calculated and the expansion level to-date is quantified, CI or embedded pin measurements can be used to monitor future expansion.

While in-plane expansion measurements via CI or embedded pins is useful for the detection and monitoring of ASR at the initial stages, an additional monitoring parameter in the through-thickness direction is required to monitor more advanced ASR progression. The difference between the in-plane expansion and the through-thickness expansion is due to the reinforcement detailing and the resulting difference in confinement between the in-plane and through-thickness direction. Through thickness expansion is less confined due to the fact that there is no reinforcement in that direction, therefore, expansion occurs preferentially in the through-thickness direction. Similarly, for unreinforced concrete backfill, expansion occurs in all directions.

Through-Thickness Expansion

The need for through-thickness expansion monitoring is triggered by a CCI exceeding 1 mm/m. The expansions of the test specimens in the MPR/FSEL large-scale test programs were significantly more pronounced in the through-thickness direction (i.e. perpendicular to the reinforcement mats) than the in-plane directions (i.e. on the faces of the specimens parallel to the reinforcement mats).

Pre-Instrument Expansion

To determine expansion to date at a location selected for instrument installation, Seabrook Station removes concrete cores at the location in which the instruments are installed and tests them for compressive strength and elastic modulus. Using the methodology from MPR-4153, the elastic modulus values are used to determine pre-instrument expansion in the through-thickness direction.

Cores removed for material property testing have the approximate dimensions of 4" diameter × 8" length and are tested in accordance with ASTM C39 for

Compressive Strength and C469 for Elastic Modulus. The cores are taken perpendicular to the reinforcement mat.

The cores are visually examined to confirm there is no mid-plane crack or edge-effect cracking.

Snap-Ring Borehole Extensometer

Seabrook Station installs Snap-Ring Borehole Extensometers (SRBEs) at the station to monitor through-thickness expansion. The MPR/FSEL program evaluated performance of the SRBEs, along with two other instrument types, in a test specimen representative of the concrete at Seabrook Station over a one-year period. The SRBE provided accurate measurements of through-thickness expansion throughout the test program and did not exhibit any problems related to reliability. The test program involved cycles of extended exposure to high temperature and humidity, which bounds the conditions expected at Seabrook Station.

The SRBE consists of a graphite rod that is held in place by an anchor placed in the borehole. Measurements are performed by using a depth micrometer to measure the distance from a reference anchor at the surface of the concrete to the end of the graphite rod. The SRBE design contains no electronics and does not require calibration. Failure of the SRBE is unlikely. In the event that an SRBE did fail (e.g., an anchor broke loose), Seabrook Station could install another SRBE nearby to the failed location and continue expansion monitoring. This will not result in significant loss of data.

A SRBE is installed in a core bore at each Tier 3 location. The elastic modulus is only determined at the time of core removal to determine pre-instrument expansion to date. Additionally, mid-plane or edge-effect cracking visually observed at the time of core removal. SRBE monitoring is conducted on a six month frequency.

Volumetric Expansion

Although the test programs identified through-thickness expansion as the most sensitive correlating parameter, ASR expansion can also be characterized in terms of volumetric expansion. To support that concrete at Seabrook Station is appropriately represented by the specimens from the large-scale test programs, NextEra Energy Seabrook also monitors volumetric expansion by using the CCI and extensometer measurements to calculate volumetric expansion at each monitoring location where an extensometer is installed. Volumetric expansion is determined at each monitoring interval (i.e., every six months for Tier 3 locations). An advantage of the volumetric expansion parameter is that it accounts for expansion in all three principal directions, which will address slight variation among in-plane expansion values at different locations throughout Seabrook Station.

ELEMENT 5 - MONITORING AND TRENDING

The progression of ASR degradation of the concrete is an important consideration for assessing the long term implications of ASR and specifying monitoring intervals. The most reliable means for establishing the progression of ASR degradation is to monitor expansion of the in situ concrete. Results of walkdowns are initially reviewed by a licensed Professional Engineer (PE) to determine whether the symptoms shown have potential to be ASR and if CCI measurements are needed.

In-Plane and Through-Thickness Expansion

For anchor capacity, NextEra Energy Seabrook uses in-plane expansion (CCI or embedded pins) to apply the results from the MPR/FSEL large-scale test program. For shear capacity, and reinforcement anchorage, NextEra Energy Seabrook uses in-plane expansion (CCI or embedded pins) and through-thickness expansion (modulus + SRBE measurements) to apply the results from the MPR/FSEL large-scale test program.

ASR is a slow progressing phenomenon. NextEra Energy Seabrook will consider the rate at which a location is approaching the established limits and take appropriate action if the limit is anticipated to be exceeded prior to the next scheduled inspection.

Volumetric Expansion

For shear capacity and reinforcement anchorage, NextEra Energy Seabrook uses volumetric expansion to compare observed ASR progression at the plant with the test specimens from the MPR/FSEL large-scale testing programs.

ASR is a slow progressing phenomenon. NextEra Energy Seabrook will consider the rate at which a location is approaching the established limits and take appropriate action if the limit is projected to be exceeded prior to the next scheduled inspection.

ELEMENT 6 - ACCEPTANCE CRITERIA

Identification of the typical symptoms indicative of ASR generates the need to initially start monitoring the area using CCI. For the structures subject to ASR monitoring, rebar strain as a result of ASR induced stresses and ASR induced stresses in combination with design bases loads will be verified to be within code allowable limits.

In-Plane Expansion for Initial Screening

A Combined Cracking Index (CCI) and corresponding in-plane expansion values are established at thresholds at which structural evaluation is necessary (see table below). The Cracking Index (CI) is the summation of the crack widths on the horizontal or vertical sides of 20-inch by 30-inch grid on the ASR-affected concrete surface. The horizontal and vertical Cracking Indices are averaged to obtain a Combined Cracking Index (CCI) for each area of interest. A CCI of less than the 1.0 mm/m can be deemed acceptable with deficiencies (Tier 2). Deficiencies determined to be acceptable with further review are trended for evidence of further degradation. The change from qualitative monitoring to quantitative monitoring occurs when the

Cracking Index (CI) of the pattern cracking equals or is greater than 0.5 mm/m (0.05% expansion) in the vertical and horizontal directions. Concrete crack widths less than 0.05 mm (0.05% expansion) cannot be accurately measured and reliably repeated with standard, visual inspection equipment. A CCI of 1.0 mm/m (0.1%) or greater requires structural evaluation (Tier 3). All locations meeting Tier 3 criteria will be monitored via CCI (in-plane expansion) and borehole extensometers (through-thickness expansion) on a ½ year (6-month) inspection. All locations meeting the Tier 2 structures monitoring criteria will be monitored on a 2.5 year (30-month) frequency. CCI correlates well with strain in the in-plane directions and the ability to visually detect cracking in exposed surfaces making it an effective initial detection parameter. Tier 1 structures do not display signs of ASR and are monitored consistent with the Structures Monitoring Program. In the event ASR monitoring results indicate a need to amend either the monitoring program acceptance criteria or the frequency of monitoring, NextEra Energy Seabrook will take such action under the Operating Experience element of the Alkali-Silica Reaction Aging Management Program.

Tier	Structures Monitoring Program Category	Recommendation for Individual Concrete Components	Criteria
3	Unacceptable (requires further evaluation)	• Structural Evaluation • Implement enhanced ASR monitoring, such as through-wall expansion monitoring using Extensometers.	1.0 mm/m (0.1%) or greater strain measurement (CCI or pin-pin)
2	Acceptable with Deficiencies	Quantitative Monitoring and Trending	• 0.5 mm/m (0.05%) or greater strain measurement (CCI or pin-pin) • CI or pin-pin measurement of greater than 0.5 mm/m (0.05%) in the vertical and horizontal directions
		Qualitative Monitoring	Any area with visual presence of ASR (as defined in FHWA-HIF-12-022) accompanied by a CI of less than 0.5 mm/m (0.05%) in the vertical and horizontal directions.
1	Acceptable	Routine inspection as prescribed by the Structural Monitoring Program	Area has no indications of pattern cracking or water ingress. No visual symptoms of ASR.

Criterion of 1mm/m (0.1%) distinguishes between Tier 2 and Tier 3 locations in relation to in-plane expansion. The large-scale test program shows agreement between embedded pins and CCI, therefore ensuring CCI is acceptable. A structural evaluation is needed when the CCI reaches what is classified as Tier 3 (CCI > 1 mm/m, in-plane expansion > 0.1%). The structural evaluation should reflect the current expansion levels of the structure.

For ASR-affected structures within the scope of the Building Deformation AMP, the structural evaluation for building deformation fulfills the requirement in the ASR AMP for structural evaluation of Tier 3 structures. For ASR-affected structures that are within the scope of the ASR AMP but not within the scope of the Building Deformation AMP, a structural evaluation that considers the effects of ASR may not exist at the time it reaches Tier 3. In such cases, it will be necessary to perform the evaluation.

If a structural evaluation has already been performed to evaluate building deformation, plant personnel will verify that the in-plane expansion included in the structural evaluation bounds the as-found condition. If necessary, the existing evaluation will be updated to bound the as-found condition and provide margin for future expansion.

It is noted that the Tiers are intended for (1) initial screening of structures, (2) determination of when to install extensometers, and (3) determination of the base monitoring frequency.

Once a structural evaluation is performed for building deformation, the monitoring frequency will be established based on the most stringent criteria. For example a Stage Two Building Deformation Evaluation that is monitored on a 18 month frequency may have Tier 3 location monitored on a six month frequency and a Stage Three Building Evaluation that is monitored on a 6 month frequency may have Tier 2 locations that will also be monitored on a 6 month frequency.

In-Plane Expansion for Anchor Bolts and Structural Attachments
A specific in-plane expansion acceptance criterion[3] was established for anchor capacity by the large-scale test program test reports, and is presented in FP#101020, Section 2.1. Maintaining this limit is assured by periodically measuring in-plane expansion in areas affected by ASR.

Through-Thickness Expansion
In areas in which the CCI is classified as Tier 3, the expansion due to ASR will be monitored in the through-thickness direction as well. Specific acceptance criteria have been established by the large-scale test program test reports, and are presented in FP#101020, Section 2.1. Maintaining these limits is assured by periodically measuring through-thickness expansion in areas affected by ASR.

Volumetric Expansion
In areas in which the in-plane expansion measurement is classified as Tier 3, the expansion due to ASR will be monitored in the through-thickness direction as well. Specific acceptance criteria have been established by the large-scale test program test reports, and are summarized in FP#101050, Appendix B. Maintaining these limits is assured by periodically measuring through-thickness expansion in areas affected by ASR.

ELEMENT 7 - Corrective Actions
Evaluations will be performed under the NextEra Energy Seabrook Corrective Action Program (CAP) and an appropriate analysis will be performed to evaluate against the design basis of that structure. The NextEra Energy Quality Assurance Program and Nuclear Fleet procedures will be utilized to meet Element 7 Corrective Actions.

ELEMENT 8 - CONFIRMATION PROCESS

The FPL/NextEra Energy Quality Assurance Program and Nuclear Fleet procedures will be utilized to meet Element 8 Confirmation Process.

ELEMENT 9 - ADMINISTRATIVE CONTROLS

The FPL/NextEra Energy Quality Assurance Program and Nuclear Fleet procedures will be utilized to meet Element 9 Administrative Controls.

ELEMENT 10 - OPERATING EXPERIENCE

The primary source of OE, both industry and plant specific, was the NextEra Energy Seabrook Corrective Action Program documentation. The NextEra Energy Seabrook Corrective Action Program is used to document review of relevant external OE including INPO documents, NRC communications and Westinghouse documents, and plant specific OE including corrective actions, maintenance work, orders generated in response

[3] Expansion Limit Criteria are considered proprietary to NextEra Energy Seabrook. FP #101020 MPR-4288, Revision 0, "Seabrook Station: Impact of Alkali-Silica Reaction on the Structural Design Evaluations," July 2016; License Amendment Request 16-03, "Revise Current Licensing Basis to Adopt a Methodology for the Analysis of Seismic Category I; Structures with Concrete Affected by Alkali-Silica Reaction," August 1, 2016

to a structure, system or component deficiencies, system and program health reports, self-assessment reports and NRC and INPO inspection reports.

Newly Identified Operating Experience (OE)
NextEra Energy Seabrook will update the Aging Management Program for any new plant-specific or industry OE. This includes ongoing industry studies performed both nationally and internationally. Research data taken from these studies will be used to enhance the ASR program, if applicable. In addition NextEra Energy Seabrook has submitted a License Amendment Request to the Commission in accordance with 10CFR50.90 to incorporate a revised methodology related to ASR material properties and building deformation analysis for review and approval. NextEra Energy Seabrook will incorporate changes related to this LAR submittal as necessary to maintain alignment of the aging management program to the current license basis.

Groundwater Operating Experience
Historically, NextEra Energy Seabrook has experienced groundwater infiltration through cracks, capillaries, pore spaces, seismic isolation joints, and construction joints in the below grade walls of concrete structures. Some of these areas have shown signs of leaching, cracking, and efflorescence on the concrete due to the infiltration. During the early 1990's an evaluation was conducted to assess the effect of the groundwater infiltration on the serviceability of the concrete walls. That evaluation concluded that there would be no deleterious effect, based on the design and placement of the concrete and on the non-aggressive nature of the groundwater.

In 2009, NextEra Energy Seabrook tested seasonal groundwater samples to support the development of a License Renewal Application. The results showed some of the groundwater to be aggressive. Ground water testing performed in November 2008 and September 2009 found pH values between 6.01 and 7.51, chloride values between 19 ppm and 3900 ppm, and sulfate values between 10 ppm and 100 ppm. Aggressive chemical attack becomes a concern when environmental conditions exceed threshold values (Chlorides > 500 ppm, Sulfates >1500 ppm, or pH < 5.5). Based on determination of aggressive ground water and observed efflorescence on the concrete surface, NextEra Energy Seabrook initiated a comprehensive review of possible effects to concrete of in-scope structures.

ASR Identification OE
In 2009, NextEra Energy Seabrook performed a qualitative walkdown of plant structures and the "B" Electrical Tunnel (Control Building portion) was identified as showing the most severe indications of groundwater infiltration. Concrete core samples from this area were removed, tested for compressive strength and modulus of elasticity, and subjected to petrographic examinations. The results showed that both compressive strength and modulus of elasticity were less than the expected values, which is symptomatic of ASR. The results of the petrographic examinations also showed that the samples had experienced Alkali-Silica Reaction (ASR).

NextEra Energy Seabrook initiated an extent of condition evaluation and concrete core samples were taken from five additional areas of the plant that showed characteristics with the greatest similarity to the "B" Electrical Tunnel. Additional concrete core samples were also taken from an expanded area around the original concrete core samples in the "B" Electrical Tunnel.

Tests on these core samples confirmed that the original "B" Electrical Tunnel core samples show the most significant ASR. For the five additional areas under investigation, final results of compressive strength and modulus testing indicate that the compressive strength in all areas is greater than the strength required by the design of the structures. Modulus of elasticity was in the range of the expected value except for the Diesel Generator, Containment Enclosure Buildings, Emergency Feedwater Pumphouse, and the Equipment Vaults, which were less than the expected value in localized areas.

Evaluation of the affected structures concluded that they are fully capable of performing their safety function but margin had been reduced. Material property results from cores removed from a reinforced concrete structure do not properly represent the actual structural performance because the structural context is lost. However, the areas are potentially subject to further degradation of material properties due to the effects of ASR.

Examination of Inaccessible Areas OE
To date, NextEra has not observed ASR in in-accessible areas greater than that observed in accessible areas and does not expect to observe such expansion in the future. In general these areas are not accessible because they are buried and have no accessible interior spaces. The environmental conditions that affect ASR development are those related to alkali transport and silica solubility.

Temperature and humidity (in this case ground water) are the most significant.
The buried concrete is subject to ground water on all sides. Most accessible areas have groundwater on one side with an adjacent interior space. This arrangement allows for flow through the concrete with an alternating wet dry surface on one side. This tends to facilitate alkali transfer and higher ASR progression has been seen in these conditions as opposed to fully or constantly wetted conditions. With respect to ambient temperature, in general the higher the temperature the more soluble the silica and the faster ASR will progress. The below grade accessible areas generally have a heated interior space that means the concrete is warmer than the surrounding backfill material. The inaccessible below grade concrete will essentially be at the constant cool temperature of the surrounding backfill material. The ambient temperature and humidity conditions are no harsher for ASR than the observable concrete and so the rates of ASR progression are bounded.

Several inaccessible areas have been inspected and results to date have confirmed instances where ASR is present. However, the levels of ASR observed were consistent with that observed in accessible areas of the plant. Typical inaccessible areas inspected include underground electrical manholes, GSU transformer foundations, GSU transformer containment structures, underground SW pipe access vault, and below grade backfill concrete.

An opportunistic Structures Monitoring inspection was conducted on the underground Unit 2 Circulating Water Pipe Access Vault on June 3, 2015. The vault was found to have been flooded to approximately 15 foot elevation. Overall the condition of the concrete was found to be in good condition and deemed acceptable. Minor cracking was present (mostly on the top surface concrete) but did not exceed 0.025 inches in width and appeared to be shallow in depth for all notable instances. No visible map cracking, dark staining or gel exudation indicative of ASR was noted.

An opportunistic Structures Monitoring inspection was conducted on concrete structures associated with the Generator Step-up Transformer Units (GSU) on March 19, 2014 and June 26, 2014 for "A" and "C", respectively. The inspections encompassed both the GSU foundation and its respective containment structure. Both pit areas that were inspected showed characteristics that are suggestive of ASR (pattern cracking, and dark staining). The indications were noted on the inside face and top of the oil containment structure only. No indications were noted on the foundations. The nominal width of the pattern cracking appeared to be less than 2 mils (0.002 inches), the minimum measurable crack width that can be reliably and accurately measured. Therefore, the ASR cracking on top of the oil containment walls are classified as ASR Tier 2 -Qualitative and being monitored on a 30-month basis. The top of the south wall at 1-ED-X-JA (GSU) has a modified CCI grid (due to size restrictions) and will be classified as ASR Tier 2- Quantitative and will be monitored on a 30-month basis as well. In addition, areas of spalled concrete were found on the GSU foundations and were promptly remediated.

Confirmation of Overall Expansion Behavior

NextEra Energy Seabrook will perform several actions to confirm that expansion behavior at the plant is consistent with the specimens from the MPR/FSEL Large-scale Test Programs. These actions assess similarity of expansion behavior in terms of trends between directions and expansion levels. The actions also include corroborating the correlation of normalized modulus versus through-thickness expansion derived from the MPR/FSEL testing against plant data. This AMP may be updated as necessary to account for any findings from these checks, which are described in the table below.

Objective	Approach	When
Ongoing Monitoring (See AMP Elements 3 through 6)		
Expansion within limits from test programs	Compare measured in-plane expansion (ε_{xy}), and through-thickness expansion (ε_z), and volumetric expansion (ε_v) at the plant to limits from test programs	Intervals as specified in AMP
Lack of mid-plane crack	Inspect cores removed from ASR-affected structures (and boreholes) for evidence of mid-plane cracks	When cores are removed to install extensometers or for other reasons.
Periodic Confirmation of Expansion Behavior		

Lack of mid-plane crack	Review of records for cores removed to date or since last assessment	Periodic assessments: • At least 5 years prior to the Period of Extended Operations (PEO) • Every 10 years thereafter
Expansion initially similar in all directions but becomes preferential in z-direction	Compare Ɛxy to Ɛz using a plot of Ɛz versus Combined Cracking Index (CCI)	
Expansions within range observed in test programs	Compare measured Ɛxy, Ɛz and Ɛv at the plant to limits from test programs to check margin for future expansion	
Corroborate modulus-expansion correlation with plant data (A secondary objective of these studies is to provide additional data to confirm that expansion behavior at the plant is comparable to the test specimens.)	For 20% of the extensometer locations: • Remove cores for modulus Compare Ɛz determined from the modulus-expansion correlation with Ɛz determined from the extensometer and the original modulus result	At least 5 years prior to PEO (initial study) and 10 years thereafter (follow-up study). A detailed explanation of this approach is provided in MPR-4273, Revision 1, “Seabrook Station - Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction” (Seabrook FP# 101050).

EXCEPTIONS TO NUREG-1800

None

ENHANCEMENTS

- The Alkali-Silica Reaction (ASR) Monitoring is being implemented
- The NextEra Energy Seabrook Structures Monitoring Program, SMPM has been revised to include Alkali-silica reaction description, aging effects, inspection criteria, acceptance criteria.
- The NextEra Energy Seabrook ASME Section XI, Subsection IWL Program ES1807.031 has been revised to include Alkali-silica reaction aging effects.

CONCLUSION

To manage the aging effects of cracking due to expansion and reaction with aggregates in concrete structures, the existing Structures Monitoring Program, B.2.1.31, and ASME Section XI, Subsection IWL Program, B.2.1.28 have been augmented by this plant specific Alkali-Silica Reaction (ASR) Aging Management Program (AMP), B.2.1.31A.

Routine inspections are performed by the Structures Monitoring and the ASME Section XI, Subsection IWL Program. Areas that have no visual presence of ASR are considered "acceptable" (Tier 1). An area with an in-plane expansion of less than 0.1% (Combined Cracking Index (CCI) of less than 1.0 mm/m) is deemed "acceptable with deficiencies" (Tier 2). An area with an in-plane expansion of greater than 0.1% (CCI of 1.0 mm/m 0.1%) is deemed "unacceptable" and requires further evaluation (Tier 3). In addition, an area that meets Tier 3 requirements will be monitored for through-thickness expansion in addition to in-plane expansion. In such areas, the through-thickness expansion and in-plane expansion values will be used to determine volumetric expansion.

Evaluations will be performed under the NextEra Energy Seabrook Corrective Action Program (CAP) and an appropriate analysis will be performed to evaluate against the design basis of that structure.

The NextEra Energy Seabrook ASR AMP provides reasonable assurance that the effects of aging of in-scope concrete structures due to the presence of ASR will be managed to ensure the structures continue to perform their intended function consistent with the current licensing basis for the period of extended operation.

B.2.1.31B BUILDING DEFORMATION

PROGRAM DESCRIPTION

The Building Deformation Aging Management Program (AMP) is a new plant specific program being implemented under the existing Maintenance Rule Structures Monitoring Program. Building Deformation is an aging mechanism that may occur as a result of other aging effects of concrete. Building Deformation at Seabrook Station is primarily a result of ASR, described in LRA section B.2.1.31A, but can also result from swelling, creep, and shrinkage. Building deformation can cause components within the structures to move such that their intended functions may be impacted.

The Building Deformation Aging Management Program uses visual inspections associated with the Structures Monitoring Program and cracking measurements associated with the Alkali-Silica Reaction program to identify buildings that are experiencing deformation. The first inspection is a baseline to identify areas that are exhibiting surface cracking. The surface cracking is characterized and documented. The first inspection identifies any local areas that are exhibiting deformation. The extent of surface cracking serves as input into an analytical model. This model determines the extent of building deformation and the frequency of required visual inspections.

For building deformation, location-specific measurements (e.g. via laser target and gap measurements) are compared against location-specific criteria to evaluate acceptability of the condition.

Structural evaluations are performed on buildings and components affected by deformation as necessary to ensure that the structural function is maintained. Evaluations of structures validate structural performance against the design basis, and may use results from the large-scale test programs, as appropriate.

Evaluations for structural deformation also consider the impact to functionality of affected systems and components (e.g., conduit expansion joints). NextEra Energy Seabrook evaluates the specific circumstances against the design basis of the affected system or component. Structural evaluations are used to determine whether additional corrective actions (e.g., repairs) to the concrete or components are required. Specific criteria for selecting effective corrective actions are evaluated on a location-specific basis.

PROGRAM ELEMENTS

The following provides the results of the evaluation of each program element against the 10 elements described in Appendix A of NUREG-1800 Rev. 1, *"Standard Review Plan for Review of License Renewal Applications for Nuclear Power Plants"*.

ELEMENT 1 - SCOPE OF PROGRAM

The NextEra Energy Seabrook Building Deformation Aging Management Program provides for management of the effect of building deformation on concrete structures and associated components within the scope of license renewal. Program scope includes components within the scope of license renewal contained in concrete structures within the scope of the Structures Monitoring Program and License Renewal ASME Section XI Subsection IWL Program. Concrete structures within the scope of this program include:

Category I Structures
- Containment Building (including equipment hatch missile shield)
- Containment Enclosure Building
- Containment Enclosure Ventilation Area
- Service Water Cooling Tower including Switchgear Rooms
- Control Building
- Control Building Make-up Air Intake Structures
- Diesel Generator Building
- Piping (RCA) Tunnels
- Main Steam and Feed Water East and West Pipe Chase
- Waste Processing Building
- Tank Farm
- Condensate Storage Tank Enclosure
- Emergency Feed Water Pump House Building, including Electrical Cable Tunnels and Penetration Areas (Control Building to Containment)
- Fuel Storage Building
- Primary Auxiliary Building including RHR Vaults
- Service Water Pump House
- Service Water Access (Inspection) Vault
- Circulating Water Pump House Building (below elevation 21'-0)
- Safety Related Electrical Manholes and Duct Banks
- Pre-Action Valve Building

Non-Category I Structures
- Intake & Discharge Transition Structure

ELEMENT 2 - PREVENTIVE ACTIONS

There are no preventive actions specified in the NextEra Energy Seabrook Structures Monitoring Program, which includes implementation of NUREG-1801 XI.S5, XI.S6, and XI.S7. These are monitoring programs only. Similarly, the Building Deformation Aging Management Program does not rely on preventive actions.

ELEMENT 3 – PARAMETERS MONITORED/INSPECTED

The Methodology Document (FP# 101196) describes a process in which ASR-affected structures[4] are initially screened for deformation and analyzed to assess the effects on structures for the self-straining loads from ASR expansion, creep, shrinkage, and swelling. Each stage of the process (i.e., Stage One, Stage Two, and Stage Three) has

[4] The Methodology Document applies to all ASR-affected structures; it is not limited to Seismic Category I structures. Thus the methodology can be used to analyze all structures within the scope of this program.

increasing levels of rigor. The analysis and evaluation of each structure may begin at any of the three stages.

The following criteria should be considered when selecting the starting stage for analysis.

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

Structures should start at Stage One if they meet all four criteria listed above. Structures should start at Stage Two if they meet two or three of the listed criteria. Structures should start at Stage Three if they meet one or none of the listed criteria.

<u>Establish Parameters Monitored and Threshold Limits</u>
As detailed in the Methodology Document, the specific locations where ASR exists in each structure and the critical areas where the margin to Licensing Basis structural design code and design basis acceptance criteria are most limiting influence the locations and types of measurements that are used to monitor each structure. Results from the structural analysis are used to identify the critical areas for meeting the acceptance criteria. Monitoring parameters, locations, frequencies, administrative limits, and threshold limits associated with the ASR-affected structure of interested are documented in the associated structural calculation.

Field inspections shall be performed to obtain observations and measurements that can be used to quantify ASR loads applied to each structure. A list of observations and measurements that may be recorded during field inspection is provided in the table below.

Parameter	Description
Cracking suspect of ASR (visual observations)	Qualitative visual observations made of cracking that exhibits visual indications of ASR and ASR-related features, using industry guidelines,
Cracking not suspect of ASR (visual observations)	Qualitative visual observations made of cracking that do not exhibit indications of ASR. These cracks may be structural (i.e. caused by stresses acting on the structure) or caused by shrinkage or other mechanisms aside from ASR.
Other structural or material distress (visual observations)	Qualitative visual observations made of structural distress, such as buckled plates, broken welds, spalled concrete, delaminated concrete, displacement at embedded plates, damage to coatings, and chemical staining.
Crack index	Quantitative measurement of in-plane cracking on a concrete structural component using the cracking index measurement

	procedure
In-plane strain rate	Quantitative measurement of length between two points installed on a concrete component using a removable strain gage. In-plane expansion is computed as the change in length between measurements recorded at different times.
Through-thickness expansion	Quantitative measurement of the thickness of a concrete component using an extensometer device. Through-thickness expansion is computed as the change in thickness between measurements recorded at different times.
Through-thickness strain rate	Calculated value based on measurements of through-thickness expansion over a period of time.
Individual crack widths/lengths	Quantitative measurement of individual crack widths using either a crack card, an optical comparator, or any other instrument of sufficient resolution. Such measurements shall be accompanied by notes, sketches, or photographs that indicate the pattern of the cracks and their length. Also included in this category are tools that quantify the change in crack widths, such as mountable crack gages, extensometers, and invar wires
Seismic isolation joints	Quantitative measurement of the width of seismic joints that separate two adjacent structures. Also included in this category are qualitative observations of distress in seals covering or filling isolation joints, such as tears, wrinkles, and bubbles.
Structure dimensions	Quantitative measurement of a structure's dimensions or the distance between two adjacent structures. Included in this category are measurements of plumbness of walls, levelness of slabs, and bowing/bending of members.
Equipment/conduit offsets	Quantitative measurement or visual observation of building deformation through the misalignment of equipment and/or the deformation of flexible conduit joints.

A document review shall be performed for each structure. Documents that are necessary to review include design drawings and design criteria. Other additional documents shall also be reviewed as needed in order to perform susceptibility evaluations. All documents reviewed shall be the latest available revision. A list of documents that may be reviewed is provided in the table below.

	Documents	Description
Review Necessary	Structural design drawings and specifications	Structural design drawings, including excavation drawings, backfill drawings, and adjacent structure drawings as needed
	Original structural design criteria	Structural design criteria, including the Updated Final Safety Analysis Report (UFSAR), documenting loads, load combinations, and strength acceptance criteria for which the structure was originally designed
	Structural design calculations	Structural design calculations documenting the underlying assumptions of the original structural design and original design demands and capacities.
Review As Needed	Construction documentation	Construction documents, drawings, and photos documenting construction stages, concrete placement, etc. This category also includes as-built drawings and survey data following construction.
	Documentation of structural and material tests	Existing documentation of testing, including petrography that has been performed on the structure or the materials of the structure.

The number of monitoring locations and the types of measurements taken will be influenced by the sensitivity of the results to the level of expansion or deformation in these regions as well as the size and shape of ASR-affected areas in the structure.

Stage One – Susceptibility Screening Evaluation:
Threshold monitoring measurements should be performed at a frequency of 36 months. Since the Stage One analyses are performed using a conservative approach based on several CI and/or pin-to-pin in-plane expansion locations and other structural deformation parameters, there will be a limited number of threshold monitoring quantitative measurements and several qualitative observation parameters. The quantitative measurements shall be compared to the corresponding specified limits from Stage One analysis evaluation. Similarly, the qualitative threshold measurements should be within the specified description and/or limits for these observations. When the observed variables are below the specified limits, the next threshold monitoring shall be performed within the monitoring frequency of 36 months. If a quantitative or qualitative observation variable approaches the corresponding specified limits, then further evaluations or structural modifications may be considered, as described in the Methodology Document and in Element 6 of this program.

Stage Two – Analytical Evaluation:
Threshold monitoring measurements should be performed at a frequency of 18 months. Quantitative measurements include in-plane expansion measurements and measurement of additional structural deformations. The quantitative threshold

variable could be from one location or from an average of several locations with similar behavior. The quantitative measurement or average of several measurements as defined by the monitoring program shall be compared to the corresponding specified limits from Stage Two analysis evaluation. Similarly, the qualitative threshold measurements should be within the specified description and/or limits for these observations. When the observed variables are below the specified limits, then the next threshold monitoring shall be performed within the monitoring frequency of 18 months. If a quantitative or qualitative observation variable approaches the corresponding specified limits, then further evaluations or structural modifications may be considered, as described in the Methodology Document and in Element 6 of this program.

Stage Three – Detailed Evaluation:
Threshold monitoring measurements should be performed at a frequency of 6 months. Quantitative measurements include CI in-plane expansion measurements, pin-to-pin in-plane expansion measurements, crack width measurements, and measurement of other structural deformation variables. The quantitative threshold variable for each region could be from one location or from an average of several locations with similar behavior. The quantitative and qualitative measurements specified for each building shall be performed within the required frequency of inspection. The quantitative measurement or average of several measurements, as defined by the structural monitoring program, shall be compared to the corresponding specified limits from Stage Three analysis evaluation. Similarly, the qualitative threshold measurements should be within the specified description and/or limits for these observations. When the observed variables are below the specified limits, then the next threshold monitoring shall be performed within the monitoring frequency of 6 months. If a quantitative or qualitative observation variable approaches the corresponding administrative limits, then further evaluations or structural modifications may be considered, as described in the Methodology Document and in Element 6 of this program.

Summary

In summary, the structural analysis process, as described in the Methodology Document, classifies ASR-affected structures into one of three categories: (1) structures with minimal amounts of deformation that do not affect the structural capacity as determined in the original design analysis (i.e. Stage One); (2) structures with elevated levels of deformation that are shown to be acceptable using Finite Element Analysis (FEA) to calculate ASR loads but still meeting the original design basis requirements when ASR effects are included (i.e., Stage Two); and (3) structures with significant deformation that are analyzed and shown to meet the requirements of the code of record using the methods described in the Methodology Document (i.e., Stage Three).

This approach is consistent with guidance in ACI 349.3R-1996 used to establish the inspection criteria for the Structures Monitoring Program. The ASR deformation categories do not necessarily correspond to the criteria used to characterize ASR cracking in structures that is discussed in LRA section B.2.1.31A. That is, a Stage

Two structure does not necessarily have ASR cracking that is classified as Tier 2. Structures will be monitored based on the most limiting parameter for monitoring from either the ASR Aging Management Program or the Building Deformation Aging Management Program. The building deformation monitoring frequency for structures for each stage is summarized in the table below.

Stage	Deformation Evaluation Stage	Monitoring Interval[5]
1	Screening assessment	3 years
2	Analytical Evaluation	18 months
3	Detailed Evaluation	6 months

As there are no published standards that include inspection frequencies for ASR-affected structures and neither ACI 318-71 nor ASME Code, Section III, Division 2 have guidance for inspecting ASR-affected structures, the monitoring frequencies in table above are based on guidelines developed for inspecting transportation structures with ASR degradation and on the relative margin to design acceptance criteria from the structural analysis described in the Methodology Document. The guidance recommends inspections from six months to 5 years depending on the age of the damage to the structure and the rate of change in degradation. The interval for recording monitoring elements for deformation for each structure can be increased to the interval in the next lower Stage (i.e., Stage Three to Stage Two and Stage Two to Stage One) if no change in measurements are observed for 3 years. Stage One structures that have shown no change in deformation for 10 years may increase the inspection interval to once every 5 years. Structures that show no evidence of building deformation will continue to be inspected with a frequency as established by the Structures Monitoring Program.

Components Impacted by Structural Deformation

With deformation, an aging effect of concern is component functionality and structural interferences. Condition walkdowns are performed with a focus on safety-related components such as pumps, valves, conduits, piping etc. The effects of deformation on plant equipment and seismic gaps will be managed through the Corrective Action Program based on input from the Structural Monitoring Program. The identification of items of interest is entered into the NextEra Energy Seabrook Corrective Action Program (CAP) to be dispositioned for impact on plant structures. Specific features to look for include, but are not limited to, the following:

- Distorted flexible couplings
- Non-parallel pipe/conduit/HVAC joints
- Gaps, distortions, or tears in seals
- Crimped tubing
- Distorted support members/structural steel
- Distorted/bent anchor bolts

[5] NextEra Energy Seabrook has the ability to apply more stringent monitoring intervals depending on the structure-specific considerations and conditions.

- Offset rod hangers
- Support members exceeding minimum clearance
- Cracked welds
- Support embedment plates – not flush with walls
- Misaligned pipe flanges
- Misaligned pipes in penetrations
- Roof membranes and weather seals degraded
- Electrical box, panel, or fitting distorted

Component specific features may indicate irreversible deformation of the affected component or irreversible plastic deformation of the structure such as rebar yielding or rebar slip. If these features are observed, then they will be documented in the corrective action process so that future monitoring walkdowns will observe the same features. Inspections of these features are in addition to the installed monitoring elements such as strain measurements and measurements of the relative deformation between structures. All of these measurements will be performed at a frequency that ensures functionality of the affected components is not lost prior to the next inspection interval. At a minimum, measurements will be taken at the frequency described the Methodology Document and summarized above.

The walkdowns will be performed in accordance with the Structures Monitoring Program and ASME Section XI, Subsection IWL Program documents. NextEra Energy Seabrook will update the walkdown guidance documents as necessary to accommodate new Operating Experience (OE) identified during the walkdowns.

ELEMENT 4 - Detection of Aging Effects

As discussed in Element 3, baseline walkdowns are performed to identify the potential effects caused by building deformation. The results of the baseline walkdowns are used to determine the key assumptions in the structural analysis. Subsequent monitoring will be performed as part of future Structures Monitoring Program (SMP) walkdowns. The recommended inspection frequencies, as defined in the Methodology Document and summarized in Element 3, will be applied in locations where symptoms of deformation are identified; otherwise, the inspection frequency will follow the requirements of the SMP. The SMP includes periodic visual inspection of structures and components for the detection of aging effects specific for that structure. The inspections are completed by qualified individuals at a frequency determined by the characteristics of the environment in which the structure is found. NextEra Energy Seabrook will consider the rate of expansion and building deformation and will take appropriate action if the structural integrity of the structure of interest and the associated components is projected to be lost prior to the next scheduled inspection.

Components Impacted by Structural Deformation

As discussed in Element 3, baseline walkdowns to identify the potential effects for equipment impacted by building deformation will at minimum frequency of two years in accordance with the Structures Monitoring Program. The SMP includes periodic visual inspection of components impacted by structural deformation for the detection of aging effects specific for that structure. The inspections are completed by knowledgeable individuals at a frequency determined by the characteristics of the environment in which the structure is found. NextEra Energy Seabrook will consider the rate of expansion and building deformation and will take appropriate action if the functionality of associated components is projected to be lost prior to the next scheduled inspection.

ELEMENT 5 - Monitoring and Trending

Once the inspection frequencies are determined as described by Element 3, visual inspections will be used to monitor and trend future building deformation. Any new indications of building deformation will be placed in the Corrective Action Program, and evaluations will be performed to determine if inspection frequencies should be changed to ensure that future effects of degradation would be identified before loss of components' intended function.

ELEMENT 6 - Acceptance Criteria

As described in the Methodology Document, the threshold factor is the design margin expressed as the amount which ASR loads can increase beyond currently measured values that are used in the calculations such that the structure or structural component will still meet the allowable limits of the code. Threshold factor is an outcome of the evaluation, not an input to the analysis methodology approach. A unique threshold factor is calculated for each building based on the available margin, and is used to establish threshold limits for structural monitoring parameters. Threshold factors may be revised based on further analysis by using additional inspection and measurement data and/or a more refined structural analysis method without reducing the code inherent margin of safety.

An administrative limit of 97% of the threshold limit is set in addition to reductions of 90%, 95%, and 100% set for Stage One, Two, and Three threshold limits, respectively. The additional 3 percent margin plus the reduction to threshold factors for Stage One and Two analyses provide time to perform additional inspections to confirm that the limits are being approached and to initiate corrective actions. When the quantitative or qualitative threshold monitoring variables reach the administrative limits further structural evaluation in accordance with procedures specified in this methodology document shall be performed to re-evaluate the structure or to consider structural modification to alleviate the concern for the approaching variable(s) to the specified limit(s).

More frequent ASR threshold monitoring may also be performed. If a structural modification approach is considered, the as-modified structure shall be evaluated using the procedures and acceptance criteria defined in this methodology document to confirm the as-modified structure meets the ASR susceptibility evaluation; and analysis shall be performed to calculate a new threshold factor for the as-modified structure.

Chemical prestressing from ASR expansion results in strain of the rebar. The codes of record combined with the analytical approaches and acceptance criteria described in the Methodology Document ensure that the behavior of ASR-affected structures remain elastic. Monitoring of ASR-affected structures against the monitoring parameters and threshold limits identified in the calculations ensures that the rebar is not strained beyond acceptable limits.

ELEMENT 7 - Corrective Actions
Structural evaluations are performed to ensure impacted structures are in compliance with the Current Licensing Basis are documented in the Corrective Action Program. The NextEra Energy Quality Assurance Program and Nuclear Fleet procedures will be utilized to meet Element 7 Corrective Actions. (Ref: LRA A.1.5 and B.1.3.)

ELEMENT 8 - Confirmation Process
The FPL/NextEra Energy Quality Assurance Program and Nuclear Fleet procedures will be utilized to meet Element 8 Confirmation Process.

ELEMENT 9 - Administrative Controls
The FPL/NextEra Energy Quality Assurance Program and Nuclear Fleet procedures will be utilized to meet Element 9 Administrative Controls.

ELEMENT 10 - Operating Experience

Building Deformation – Containment Enclosure Building (CEB)
In late 2014, a walkdown was performed to investigate a concern from the NRC that water, leaking from SB-V-9, was leaking into the Mechanical Penetration (Mech Pen) area though building seals. The walkdown documented that a Mechanical Penetration area seal was found torn. The damaged seal was a vertical seismic gap seal between the Containment Enclosure Building (CEB) and the Containment Building (CB). It was then stated that the condition of the seal and other local evidence indicated that the damage to the seal appeared to be caused by relative building movement and not seal degradation (i.e. shrinkage or material deterioration).

Following the discovery mentioned above, Engineering identified that the damage to the seal was caused by CEB outward radial deformation. NextEra Energy Seabrook engaged an engineering firm to perform visual assessments of accessible areas surrounding the CEB to determine the behavior of the CEB, whether the CEB movement is localized or widespread, and if other plant structures or components had been impacted. A Cause and Effect Diagram was prepared to understand the physical phenomena occurring with the CEB. Parametric studies using a linear finite element model of the CEB with boundary conditions modeling parameters appropriate for estimating structural deflections and deformed shapes were performed. The results were compared to in-situ field measurements taken between structures and at seismic isolation joints between various structures. The deformation patterns simulated by finite element analysis (FEA) were generally similar to field measurements. The results of the FEA showed that the deformation of the CEB was most likely due to Alkali-Silica Reaction (ASR) expansion in the concrete when combined with the expected creep and swelling of the concrete.
The root cause to the event was determined to be the internal expansion (strain) in the CEB concrete produced by ASR in the in-plane direction of the CEB shell and ASR expansion in the backfill concrete coincident with a unique building configuration. The Root Cause Evaluation identified that there are many different symptoms of building deformation. These include:

- Conduit, duct, or piping seismic connection deformation
- Gate or door misalignment
- Seismic gap seal degradation
- Seismic gap width variations
- Fire seal degradation

(Note: above list is not intended to be all inclusive)

As a result walkdowns were performed to identify the above symptoms that may have been missed during the Structures Monitoring Program Walkdowns that were conducted prior to this discovery. The items identified were entered NextEra Energy Seabrook's Corrective Action Program.

Building Deformation – RHR & FSB
NextEra Energy Seabrook has evaluated the observations of expansion resulting in building deformation in the Residual Heat Removal (RHR) Equipment Vault and the Fuel Storage Building (FSB).

As a result of the identified observations, additional monitoring has been established in the Residual Heat Removal Vaults (i.e. invar rod extensometers and crack gauges) and enhanced use of laser measurements is being evaluated for use in the Fuel Storage Building.

Both structures were ranked as having a high potential of being affected by building deformation due to ASR. Both structures have been evaluated in accordance with the Methodology Document and were evaluated as Stage Three structures with a corresponding 6-month inspection/monitoring frequency.

Building Deformation – "B" Electrical Tunnel
NextEra Energy Seabrook has evaluated the "B" Electrical Tunnel in accordance with the Methodology Document. With the ASR loadings, the governing failure mode is out-of-plane shear of the first story North and South tunnel walls. Formation of flexural cracking at mid-height and over the visible face of these walls would precede formation of the through-thickness inclined shear cracks (before shear capacity is reached). No horizontal cracking in this vicinity of the walls have been observed, so the walls are currently not loaded to the level to cause cracking. Reference the respective Prompt Operability Determination (POD), AR 02215578.

As a result of the evaluation, an enhanced monitoring frequency has been established for select portions of the "B" Electrical Tunnel.

Building Deformation – CEVA North Wall
The lower portion of the CEVA Structure North Wall between elevation (+) 3ft and elevation (+) 19 feet exhibits extensive cracking and out-of-plane deformation (bowing). This condition is due to the expansion of the concrete fill that is below the floor slab at elevation (+) 12ft of the CEVA Structure and south of the North wall between (+) 3 ft and elevation (+) 19 ft. Based on the deformation and cracking

observed, the wall cannot be qualified to ACI 318-71. A validation study was performed on the wall to characterize the potentially delamination that is occurring. This consisted of performing Impact Echo Testing and extracted partial depth cores.

As a result of the evaluation, enhanced monitoring has been established and Engineering Change is being developed for a structural retrofit to restore the wall back to be in compliance with ACI 318-71.

Building Deformation - Safety-Related Electrical Manholes

Safety Related Electrical Manholes (EMH) W01, W02, W09, W13 through W16 were analyzed in accordance with the LAR 16-03 methodology, which includes the ASR loadings. The evaluation showed that the manhole structures would not meet the acceptance criteria in ACI 318-71, including ASR demands which were further increased by threshold factors to account for potential future ASR expansion, with the site design criteria surcharge load of 500 psf included.

Based on the evaluations the surcharge loadings within 8 feet of EMH W13 through W16 will be kept below 200 psf through physical and administrative controls.

Plant Specific Operating Experience

AR 02044627 notes that the as-measured width of seismic isolation gaps is less than the nominal value of 3 inches specified on concrete drawings for isolation between structures. There are a total of 93 as-measured gaps less than 3 inches between the following abutting structures: Containment Building, Containment Enclosure Building, Mechanical Penetration Area, West Main Steam and Feed Water Pipe Chase, Electrical Penetration Area and Emergency Feed Water Pump House. Initial finite element analysis completed determined that the deformation is attributed to ASR expansion and creep. The compensatory measure implemented requires measuring seismic isolation gaps every six months.

AR 2114299 documents that a seismic isolation joint located on an expansion boot near ductwork in the Containment Enclosure Building is vertically misaligned by approximately 2". The boot appeared to be in good shape; it was not dry or cracking. The AR determined that the cause of the misalignment is building deformation of the Containment Enclosure Building. The engineering evaluation concluded that the displaced ducts resulted in some slipping of the expansion joint material relative to the clamp at the areas of highest relative movement and that there is reasonable assurance that the joint material would most likely slip rather than tear or elongate during a seismic event. The condition was found acceptable as is and no loss of intended function was identified.

AR 02107225 documents a deformed and misaligned flexible coupling on a conduit located in the West Pipe Chase area. Based on a field walkdown, the coupling was misaligned by 1.75" which is greater than the established 1.25" acceptable limit. The cause of the misalignment was building deformation. Therefore, engineering analysis was performed to ensure that the enclosed cable can continue to perform its safety function. Even though the cable could continue to perform its safety function, the flexible conduit was repaired to restore design margin.

AR 02129621 documents the seismic gap between Containment and the CEB horizontal cantilevered concrete shield block at Azimuth 230 elevation 22' is less the minimum required seismic gap of .277 inches. The cause of the reduced gap was building deformation. An engineering analysis was performed to ensure that the structural remains operable while steps are taken to restore to design requirements.

Newly Identified Operating Experience (OE)
NextEra Energy Seabrook will update the Aging Management Program for any new plant-specific or industry OE. This includes ongoing industry studies performed both nationally and internationally. Research data taken from these studies will be used to enhance the Building Deformation Aging Management Program, if applicable. In addition NextEra Energy Seabrook has submitted a License Amendment Request to the Commission in accordance with 10CFR50.90 to incorporate a revised methodology related to ASR material properties and building deformation analysis for review and approval. NextEra Energy Seabrook will incorporate changes related to this LAR submittal as necessary to maintain alignment of the aging management program to the current license basis.

EXCEPTIONS TO NUREG-1800
None

ENHANCEMENTS
The station's Structures Monitoring Program, SMPM has been revised to include building deformation aging effects, inspection criteria, and acceptance criteria.

CONCLUSION
To manage the aging effects of building deformation due to ASR, swell, creep, and expansion, the existing Structures Monitoring Program and ASME Section XI, Subsection IWL Program, have been augmented by this plant specific Building Deformation Aging Management Program. This program:

- Characterizes the extent of deformation associated with each ASR-affected structure,
- Analyzes the structural adequacy of affected structures,
- Determines the projected rate of future deformation,
- Defines monitoring parameters, locations, thresholds and inspection frequencies to ensure that structural adequacy is maintained and that the plant has ample time to implement corrective action before structural adequacy is lost.

Establishes monitoring frequencies to ensure the functionality of components associated with ASR-affected structures is not lost prior to the next inspection interval.

Enclosure 3 to SBK-L-18072

Seabrook Station Clarifications to
SBK-L-17155 Enclosure 1, Appendix A

- **SBK-L-17155, Enclosure 1 Appendix A Page 1: first paragragh is revied as follows**

This appendix provides a detailed discussion of the technical basis for correlating results from the Shear and Reinforcement Anchorage Test Programs to the condition of reinforced concrete at Seabrook Station that has been affected by alkali-silica reaction (ASR). In particular, this appendix discusses the rationale for establishing monitoring parameters for through-thickness expansion and volumetric expansion ~~and not in-plane expansion~~. Additionally, this appendix discusses the observation that the maximum apparent in-plane expansion at Seabrook Station is slightly greater than in-plane expansion of the MPR/FSEL test specimens.

- **SBK-L-17155, Enclosure 1 Appendix A Page 12: the following section is provided prior to the "*Additional Comment on Compression*" section**

Additional Comment on Rebar Strain

Chemical pre-stressing from ASR expansion results in strain of the rebar. The codes of record combined with the analytical approaches and acceptance criteria in the Methodology Document (FP# 101196) ensure that the behavior of ASR-affected structures remain elastic.

- Containment is designed in accordance with ASME Section III, Division 2. The stress and strain limits for reinforcing steel under service and factored loads are as specified in ASME Sec. III, Div. 2.

- Other seismic Category I structures that were designed to ACI 318-71 (i.e., Seismic Category 1 structures other than Containment) are analyzed using approaches described in the Methodology Document and meet the acceptance criteria therein will respond elastically under realistic (un-factored) normal operating or service load conditions. This conclusion is based on parametric studies and review of calculation results for a sample of Seabrook Station Category I structures, as discussed in SBK-L-17204.

The calculations that evaluate ASR-affected structures in accordance with the Methodology Document identify parameters to monitor and associated thresholds such that a structure remains bounded by the analysis and responds elastically.

Enclosure 4 to SBK-L-18072

MPR-4153, Revision 3, "Seabrook Station – Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction," July 2016 (Seabrook FP# 100918)

(Non-proprietary)

Non-Proprietary Version

MPR

MPR-4153
Revision 3
(Seabrook FP # 100918)
September 2017

Seabrook Station - Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction

QUALITY ASSURANCE DOCUMENT
This document has been prepared, reviewed, and approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B and/or ASME NQA-1, as specified in the MPR Nuclear Quality Assurance Program.

Prepared for

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MPR

Seabrook Station - Approach for Determining Through-Thickness Expansion from Alkali-Silica Reaction

MPR-4153
Revision 3
(Seabrook FP # 100918)

September 2017

QUALITY ASSURANCE DOCUMENT
This document has been prepared, reviewed, and approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B and/or ASME NQA-1, as specified in the MPR Nuclear Quality Assurance Program.

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RECORD OF REVISIONS

Revision	Affected Pages	Description
0	All	Initial Issue
1	Body of Report, Appendix A	Included corrected data for expansion of FSEL test specimens in the through-thickness direction. Also made minor editorial changes throughout the body of the report.
2	Body of Report, Appendix A, and Appendix D	Updated to include final test program results. Included expansion data for all FSEL test specimens in the through-thickness direction and data for the first campaign of extensometer locations at Seabrook Station. Expanded discussion of uncertainty. Also made minor editorial changes throughout the body of the report.
3	Body of Report, Appendix A, and Appendix D	Updated to include additional literature data and expansion information from additional extensometer locations at Seabrook Station. Also made minor editorial changes throughout the body of the report.

Executive Summary

This report recommends a methodology for determining the extent of through-thickness expansion of reinforced concrete structural members at Seabrook Station. Quantifying through-thickness expansion enables NextEra to apply the results of the structural testing programs to Seabrook Station based on the condition of existing plant structures and ensure that action is taken before expansion at Seabrook Station exceeds the bounds of the testing programs.

Data from the structural testing programs show that expansion in the in-plane direction plateaus at low expansion levels, while expansion in the through-thickness direction continues to increase as alkali-silica reaction (ASR) proceeds. Accordingly, the test programs provide results that correlate structural performance to expansion in the through-thickness direction.

NextEra has installed instruments (i.e., extensometers) in concrete structures at Seabrook Station to measure expansion in the through-thickness direction that occurs after time of installation through the end of plant life. To calculate total expansion, NextEra needs to determine expansion from original construction until the time the extensometer is installed (pre-instrument expansion).

MPR recommends the following approach for determining total ASR-induced through-thickness expansion at each instrumented location at Seabrook Station. The recommended method determines the pre-instrument expansion based on the observed reduction in modulus of elasticity.

1. Determine the current elastic modulus of the concrete by material property testing of cores removed from the structure. Elastic modulus testing requires companion compressive strength testing. As a result, MPR recommends obtaining a minimum of four test specimens at each proposed monitoring location. Two test specimens are for compressive strength testing and two test specimens are for subsequent elastic modulus testing.

2. Establish the original elastic modulus of the concrete by either (1) using the ACI 318-71 correlation to calculate elastic modulus from 28-day compressive strength records or (2) obtaining cores from representative ASR-free locations and testing for elastic modulus.

3. Calculate the reduction in elastic modulus by taking the ratio of the test result from the ASR-affected area to the original elastic modulus.

4. Quantify through-thickness expansion from original construction to the time the extensometer is installed using the correlation developed in this report. The correlation relates reduction in elastic modulus with measured expansion from beam specimens used during the large-scale ASR structural testing programs and provides a conservative estimate of pre-instrument expansion levels at Seabrook Station.

5. Calculate total expansion levels by adding the extensometer measurements to the expansion at the time of instrument installation.

Contents

Tables

Figures

1 Introduction

1.1 Purpose

This report recommends a methodology for determining the extent of through-thickness expansion of reinforced concrete structural members that are affected by alkali-silica reaction (ASR) at Seabrook Station. Quantifying through-thickness expansion of existing plant structures is necessary to relate the extent of ASR in a given structure to the results of the structural testing programs at Ferguson Structural Engineering Laboratory (FSEL).

The methodology recommended in this report is part of determining if expansion levels at Seabrook Station are within the limits of the test programs.

Revision 3 of this report incorporates data through September 2017, and includes all planned extensometer locations. Seabrook Station has implemented the recommended methodology and the pre-instrument expansion (i.e., expansion to-date) associated with ASR-affected extensometer locations has been determined. All locations evaluated at Seabrook Station as of September 2017 are within the limits of the test programs. Results are documented in Section 5 of this report and in Appendix D (Reference 27).

1.2 Background

1.2.1 Overview of Alkali-Silica Reaction

ASR occurs in concrete when reactive silica in the aggregate combines with alkali ions (Na^+, K^+) in the pore solution. The reaction produces a gel that expands as it absorbs moisture, exerting tensile stress on the surrounding concrete and resulting in cracking. Typical cracking caused by ASR is described as "pattern" or "map" cracking and is usually accompanied by dark staining adjacent to the cracks. Figure 1-1 provides an illustration of this process.

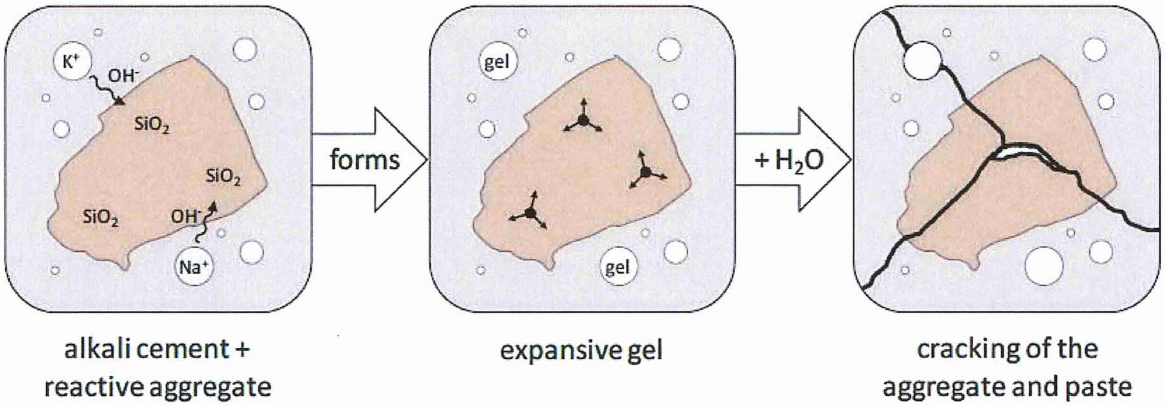

Figure 1-1. ASR Expansion Mechanism

Several publications indicate that the cracking may degrade the material properties of the concrete (References 1, 2, and 3). The concrete properties most rapidly and severely affected are the elastic modulus and tensile strength. Compressive strength is also affected, but less rapidly and less severely.

While development of ASR causes a reduction in material properties, there is not necessarily a corresponding decrease in structural performance. As discussed in previous MPR reports on ASR at Seabrook Station and the approach for the test programs (Reference 4 and Reference 5), cores removed from a reinforced ASR-affected structure are no longer confined by the reinforcement and do not represent the structural context of the in-situ condition. Therefore, material properties obtained from cores have limited applicability for evaluating the capacity of a structure.

1.2.2 ASR at Seabrook Station

NextEra has identified ASR in multiple safety-related, reinforced concrete structures at Seabrook Station (Reference 6). MPR performed a structural assessment (Reference 4) of selected ASR-affected structures to evaluate their adequacy given the presence of ASR. Based on the low level of observed cracking and the apparent slow rate of change, MPR concluded that these structures are suitable for continued service for at least an interim period (i.e., at least several years).

The interim structural assessment considered the various limit states for reinforced concrete and applied capacity reduction factors derived from test data in publicly available literature. Based on the lack of representative literature data, MPR executed large-scale test programs (MPR/FSEL test programs) to evaluate shear capacity, reinforcement anchorage, and anchor bolt capacity of ASR-affected reinforced concrete.

Follow-up evaluations are assessing the long-term adequacy of the concrete structures at Seabrook Station. The evaluations account for the impact of ASR on structural capacity and structural demands. Results from the large-scale test programs performed at FSEL using test specimens that were specifically designed and fabricated to represent reinforced concrete at Seabrook Station will be used for the analyses.

1.2.3 MPR/FSEL Test Programs

MPR sponsored four test programs at FSEL to support NextEra's efforts to resolve the ASR issue identified at Seabrook Station. The MPR/FSEL test programs were designed to ensure the test results are applicable to the range of structures at Seabrook Station. Three of the test programs focused on the structural performance data necessary to complete a definitive assessment of ASR-affected structures. The fourth test program evaluated instruments for monitoring expansion of structures at Seabrook Station. A brief overview of each test program is provided below.

- Anchor Test Program: This program evaluated the impact of ASR on performance of anchors installed in the concrete. Tests were performed at multiple levels of ASR degradation.

- Shear Test Program: This program evaluated the impact of ASR on shear performance of reinforced concrete beams. Tests were performed at multiple levels of ASR degradation.

- Reinforcement Anchorage Test Program: This program evaluated the impact of ASR on reinforcement anchorage using beams that had reinforcement lap splices. Tests were performed at multiple levels of ASR degradation.

- Instrumentation Test Program: This program evaluated instruments for measurement of through-thickness expansion. Insights gained from this program were used to select which instrument to use at Seabrook Station and to refine installation procedures.

As part of the test programs, FSEL monitored development of ASR. For the Shear, Reinforcement Anchorage, and Instrumentation Test Programs, FSEL measured expansion of the test specimens and determined the effect on material properties of concrete, which are related to ASR development. Using this information, this report recommends a methodology for determining the extent of ASR-induced expansion at Seabrook Station. (Similar data were not obtained as part of the Anchor Test Program, so this report does not utilize expansion data from the Anchor Test Program.) Quantifying the extent of ASR development will enable comparison of the test data to the condition of existing structures at Seabrook Station.

Testing was conducted under FSEL's project-specific quality system manual with technical and quality assurance oversight from MPR. MPR commercially dedicated the testing services performed by FSEL. Commercial grade dedication of services from the test programs relevant for this report is documented in Reference 22 and Reference 26 and presented in Reference 5 unless noted in Appendix A.

2 Expansion Behavior in Test Specimens

This section discusses expansion behavior observed in the test specimens and the implications for monitoring ASR development in structures at Seabrook Station. An overview of test specimen design is included to provide context for understanding the observed expansion behavior.

2.1 Overview of Test Specimens

2.1.1 Reinforcement Pattern

The MPR/FSEL test program specimens were large, reinforced concrete beams. Most test specimens were [redacted] feet-[redacted] inches long, [redacted] inches wide, and [redacted] inches thick (References 7.1 and 7.2). The test specimens were designed to represent the configuration of reinforced concrete structural members at Seabrook Station. In particular, the test area of each test specimen included two-dimensional reinforcement mats on two opposite faces, which is the same reinforcement detailing used for most reinforced concrete buildings at Seabrook Station (e.g., walls that have reinforcement mats on the interior and exterior faces). Figure 2-1 provides a schematic of the reinforcement pattern in an example shear test specimen (Reference 7.3). The reinforcement anchorage and instrumentation test specimens had some design differences (e.g., [redacted], but all test specimens contained two-dimensional reinforcement mats consistent with the example in Figure 2-1 (References 7.4 and 7.5).

[redacted]

Figure 2-1. Example Reinforcement Pattern in Shear Test Specimen (Reference 7.3)

2.1.2 Expansion Measurements

The methods for monitoring expansion in shear and reinforcement anchorage test specimens included crack indexing, mechanical measurements of reference points embedded in the concrete during fabrication (embedded pins), and measurement of the expansion profile across the test specimen height using a custom frame (z-frame).

[redacted] embedded pins were used to characterize in-plane expansion. As ASR occurred, the concrete between a given set of pins expanded, and the distance between the pins increased. Measurements were taken at both the backside and the inside faces[1] of each test specimen, [redacted] in the perpendicular direction and [redacted] in the longitudinal direction, as shown in Figure 2-2. For each direction, the [redacted] expansion values on each face were averaged to obtain the percent expansion in that direction for that face.

[redacted]

Figure 2-2. In-Plane Expansion Measurement Using Embedded Pins (Reference 5)

A custom frame (i.e., z-frame) designed and fabricated by FSEL was used to assess expansion in the through-thickness direction. The z-frame (Figure 2-3) contacted the test specimens at [redacted] on formed concrete surfaces and was aligned to both ends of the [redacted] pins embedded for through-thickness measurements. The arrangement allowed for a total of [redacted] measurements to be taken using a calibrated depth micrometer, [redacted] from each side at corresponding locations across the inside and backside faces. Measurements from the z-frame allowed the thickness of expanded test specimens to be calculated at [redacted] locations such that the profile of the expanded test specimen could be determined. The average of thickness readings from all [redacted] locations was used.

[1] The top and bottom of the test specimens are referred to as the "backside" face and "inside" face, respectively, and correspond to the exterior and interior surfaces of a wall at Seabrook Station.

[redacted]

Figure 2-3. Through-Thickness Expansion Measurements Using the Z-frame (Reference 5)

Prior to adopting the z-frame, through-thickness expansion was monitored using embedded pins, shown in Figure 2-4. [redacted] measurement of the through-thickness expansion was taken per face using the [redacted] embedded pins. For test specimens that were tested before the z-frame methodology was adopted, the through-thickness expansions measured using embedded pins were adjusted using the relationship described in Reference 5.[2] The difference between the z-frame methodology and the embedded pins methodology is that the gage length of the z-frame is the full [redacted]-inch thickness of the specimen, whereas the gage length of the embedded pins is only [redacted] inches. The relationship in Reference 5 accounts for the sensitivity of the percent expansion to gage length when expansion is concentrated in a single, large longitudinal crack.

For the instrumentation specimen, through-thickness expansion was monitored using a depth gage inserted into small bore holes that go completely through the specimen.

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[2]All through-thickness expansion values associated with shear and reinforcement anchorage test specimens presented in the this report are either expansion values obtained directly from the z-frame or were estimated from embedded pin measurements and Equation 5-1 in Reference 5.

2.2 EXPANSION IN REINFORCED CONCRETE

2.2.1 Test Specimens

Expansion of the test specimens was significantly more pronounced in the through-thickness direction (i.e., perpendicular to the reinforcement mats) than the in-plane direction (i.e., on the faces of the specimens parallel to the reinforcement mats). Expansion in the in-plane direction plateaued at low levels, while expansion in the through-thickness direction continued to increase. This behavior can be seen in Figure 2-4, which is a plot of expansion for Specimen [redacted] based on monitoring the distance between the embedded rods.[3] Expansion behavior in this test specimen is representative of other test specimens.

[redacted]

Figure 2-4. Expansion Trends in Example Test Specimen

The difference between in-plane expansion and through-thickness expansion is due to the reinforcement detailing and the resulting difference in confinement between the in-plane and through-thickness directions. The reinforcement mats confined expansion in the in-plane direction. Through-thickness expansion, on the other hand, was not confined because there was

[3] Figure 2-4 is for illustrative purposes only. Periodic monitoring of expansion is considered for information only, whereas the measurements at the time of testing are formal test measurements.

no reinforcement in that direction. Therefore, expansion occurred preferentially in the through-thickness direction.

2.2.2 Literature Review

The observed preferential expansion in the through-thickness direction is consistent with literature on ASR-induced expansion (References 2 and 9). Literature suggests that when reinforcement is present to restrain the tensile force exerted by ASR-induced expansion, an equivalent compressive force develops in the concrete, which creates a prestressing effect. If tensile loads are applied to the structure, the compressive stresses in the concrete from prestressing must be overcome before there is a net tensile stress.

2.3 IMPLICATIONS FOR MONITORING ASR AT SEABROOK

Based on the expansion behavior observed in the test specimens, expansion in the through-thickness direction is a more sensitive indicator of ASR development than in-plane expansion for concrete elements with reinforcement mats but no through-thickness reinforcement. In-plane expansion is a readily available parameter that can be used to assist with diagnosis of ASR-affected reinforced concrete. However, because confinement restrains in-plane expansion at a relatively low level, in-plane expansion is not an adequate monitoring parameter by itself. Accordingly, the results of the Shear and Reinforcement Anchorage Test Programs were correlated to expansion in the through-thickness direction.

NextEra has installed instruments (i.e., extensometers) in concrete structures at Seabrook Station to monitor expansion in the through-thickness direction. Instruments were installed in ASR-affected areas and in some areas unaffected by ASR. The instruments in areas unaffected by ASR provide a reference measurement to gauge effects, such as thermal expansion, that could influence the ASR-induced expansion measurements.

The instruments measure through-thickness expansion that occurs after the instrument is installed. To determine the cumulative expansion since original construction, this expansion measurement must be added to the expansion up to the time the instrument is installed.
The subsequent sections of this report provide a methodology for determining the pre-instrument expansion.

3 Determining Pre-Instrument Expansion from Elastic Modulus

This section describes the technical basis and methodology for using the reduction in elastic modulus to quantify the total ASR-induced expansion to-date in the through-thickness direction prior to instrument installation (pre-instrument expansion). The methodology depends on determining the elastic modulus at the time of instrument installation from cores and establishing the original elastic modulus to provide a point of reference. The original elastic modulus may be determined by testing reference cores from concrete without symptoms of ASR or by using original construction data with an ACI correlation that relates compressive strength to elastic modulus.

Specific topics discussed in this section include:

- Evaluation of changes in material properties that indicate ASR-induced expansion,
- Development of the correlation between expansion and elastic modulus based on test data from the MPR/FSEL test programs, and
- Determination of the original elastic modulus at Seabrook Station, which is used as the point of reference for determining reduction in elastic modulus.

The discussion in this section relies on test results obtained from the large-scale ASR testing programs at FSEL (Reference 5).

The correlation between normalized elastic modulus and through-thickness expansion presented in this section determines best-estimate pre-instrument expansion values for concrete structures at Seabrook Station. As discussed in Section 4, a normalized modulus reduction factor of [redacted] is applied so that the final calculated through-thickness expansion is conservative.

3.1 Material Properties of Test Specimens

As part of the MPR/FSEL test programs, FSEL obtained material property data on the test specimens at different levels of ASR-induced expansion. The difference between the 28-day material property result and the material property result at the time of testing may be used to quantify development of ASR.

3.1.1 Material Property Testing during FSEL Structural Testing Programs

During fabrication of the test specimens, FSEL prepared cylinders (approximately 8 inches in height and 4 inches in diameter) using the same batch of concrete as the test specimens (FSEL Procedure 1-5, Reference 5). A subset of these cylinders were tested 28 days after fabrication to provide initial values for the material properties of the specimen, including compressive strength, elastic modulus, and splitting tensile strength (Reference 12). At the time of load testing a shear or reinforcement anchorage specimen, FSEL obtained cores from the specimen and performed testing for material properties. For the instrumentation specimen, FSEL obtained cores and performed material property testing at selected expansion levels.

The 28-day cylinders were fabricated in accordance with ASTM C31-10 and C192-07 (FSEL Procedure 1-5, Reference 5). Cores for material property testing were obtained in accordance with ASTM C42-12. Compressive strength testing was conducted in accordance with ASTM C39-12 (FSEL Procedure 5-3, Reference 5); elastic modulus testing was performed in accordance with ASTM C469-10 (FSEL Procedure 5-4, Reference 5), and splitting tensile testing was carried out in accordance with ASTM 370-12 (FSEL Procedure 5-5, Reference 5).

Data from all ASR-affected test specimens were included in MPR's evaluation. Data from control test specimens were not included.

3.1.2 Compressive Strength and Elastic Modulus

Figure 3-1 is a plot showing the normalized values for compressive strength and elastic modulus as a function of expansion (Reference 13; Appendix A). A normalized material property is the ratio of the property at the time FSEL obtained the expansion measurement divided by the material property obtained from testing a cylinder 28 days after fabrication.

[redacted]

Figure 3-1. Compressive Strength and Elastic Modulus as a Function of Through-Thickness Expansion from Test Data (Reference 13)

Key observations from Figure 3-1 include the following:

- Normalized elastic modulus follows a trend where elastic modulus decreases sharply at expansion levels less than about [redacted]%. The trend indicates a more gradual decrease at higher expansion levels.

- Normalized compressive strength shows a general decreasing trend with increasing expansion levels; however, compared to elastic modulus, there is lower sensitivity with expansion (i.e., the slope is shallower) and there is more data scatter.

Literature data indicate that trends for the normalized material properties discussed above are consistent with the material property results from an array of test programs (References 1 and 2). In particular, the literature concludes that reduction in elastic modulus is more sensitive to ASR development than compressive strength.

3.1.3 Splitting Tensile Strength

Figure 3-2 shows the splitting tensile strength values as a function of through-thickness expansion. Normalized splitting tensile strength results (which require a 28-day value) are not available because the test programs did not start obtaining these results until May 2014, after FSEL had fabricated many of the test specimens. In addition, FSEL Procedure 5-6 (Reference 5) allows for omission of splitting tensile tests on cores due to the difficulty in extracting testable cores from members with significant cracking due to ASR. Using this provision, splitting tensile strength was not performed on cores from [redacted]. Similarly, only two cores from [redacted] and one core from [redacted] were tested (Reference 5).

[redacted]

Figure 3-2. Splitting Tensile Strength as a Function of Through-Thickness Expansion from Test Data (Reference 13)

As shown above, splitting tensile data from higher expansion levels have approximately the same splitting tensile strength values as data from low expansion levels. Even if normalized data were available, sensitivity with expansion would still be low (i.e., shallow slope). Accordingly, MPR concludes that a correlation to expansion using normalized tensile strength is unlikely to be more sensitive than a correlation using normalized elastic modulus.

3.2 Development of Correlation between Modulus and Expansion

3.2.1 Data from MPR/FSEL Test Programs

Figure 3-3 includes a plot of the test data for reduction in modulus of elasticity and the corresponding through-thickness expansion measurements (Reference 13; Appendix A).
The plot uses a normalized modulus value that is the ratio of the elastic modulus at the time the expansion measurement was obtained (E_t) divided by the 28-day elastic modulus (E_o).

[redacted]

Figure 3-3. Elastic Modulus as a Function of Through-Thickness Expansion from Test Data (Reference 13)

Results of calculations using the data from Figure 3-3 include the following:

- The correlation shown in Figure 3-3 has the following equation determined by [redacted] least-squares regression (Reference 13; Appendix A):

[redacted] [Equation 1]

- The correlation fits well with the data and therefore supports use of a [redacted] formulation. The coefficient of determination (R^2) is [redacted] (Reference 13; Appendix A). MPR performed scoping evaluations of several different forms of the equation for the correlation and determined that a [redacted] formulation [redacted] provided the best fit.

3.2.2 Data from Literature

As part of the Reference 13 calculation, MPR compared the relationship developed from the FSEL test data against data available in literature (References 14, 15, 16, 28, 29, 30, and 31) in Figure 3-4. The literature data reflect small specimens that were cast and cured as unconfined concrete.

[redacted]

Figure 3-4. Comparison of Derived Relationship with Literature Data (Reference 13)

Overall, the trend from the literature data compares favorably with the correlation generated from the FSEL data. Accordingly, the comparison to literature data corroborates application of the experimentally-determined correlation at Seabrook Station.

3.2.3 Applicability of Correlation to Seabrook Station

The correlation developed from the FSEL data relating expansion to reduction in elastic modulus is applicable to reinforced concrete structures at Seabrook Station. The test data used to generate the correlation were obtained from test specimens that were designed to be as representative as practical of the concrete at Seabrook Station, including the reinforcement detailing. Additionally, comparison against literature data shows that the correlation follows a trend that is consistent with other published studies which cover a range of concrete mixtures.

3.3 Establishing Original Elastic Modulus at Seabrook

The correlation shown in Figure 3-3 and provided in Equation 1 uses the 28-day elastic modulus as an input for determining expansion. However, consistent with typical construction practices, material property testing of concrete used at Seabrook Station verified only the 28-day compressive strength; the elastic modulus was not measured. This section describes two approaches for establishing the 28-day elastic modulus for concrete at Seabrook Station.

MPR notes that there are differences between the original elastic modulus data used to generate Equation 1 and data that will be used to determine pre-instrument expansion at Seabrook Station. These differences are assessed in Section 4.2.

3.3.1 Approach 1: Code Equation Based on Compressive Strength

ACI 318-71 (Reference 17) provides the following equation for the elastic modulus of concrete (E_c) calculated based on compressive strength (f_c') and the density of concrete in lb/ft^3 (w_c):

$$Ec = 33 \times w_c^{1.5} \times \sqrt{(fc')} \qquad \text{[Equation 2]}$$

The equation presented in ACI 318-71 is based on fitting a curve to publicly available information on compressive strength and elastic modulus of various concrete specimens. The data used cover a range of concrete mixtures from lightweight concrete to normal weight concrete.

Confirmation of Code Equation for FSEL-Generated Data

Using data from the MPR/FSEL test programs for 28-day compressive strength and elastic modulus for a concrete mix design that represented Seabrook Station, MPR confirmed that the ACI equation is applicable (Reference 18; Appendix B). ACI 318-71 states that the actual elastic modulus is expected to be within ±20% of the calculated value. As shown in Figure 3-4, [redacted] of [redacted] data points ([redacted]%) obtained from the test programs met this criterion.

[redacted]

Figure 3-5. Comparison of Test Data to ACI Equation (Reference 18)

MPR concludes that the ACI 318-71 equation is applicable for concrete at Seabrook Station for the following reasons:

- The FSEL data are consistent with the equation from ACI 318-71 and the stated variance of ±20%.
- The concrete test specimens fabricated by FSEL were designed to be representative of the concrete used at Seabrook Station and therefore better represent the concrete at Seabrook than the range of mixtures used to generate the code equation.

Original Compressive Strength

Using original construction records for compressive strength tests and the ACI 318-71 correlation, NextEra could establish the 28-day elastic modulus.

NextEra has retrieved records for concrete fabrication from original construction for selected buildings. For convenience, MPR Calculation 0326-0062-CLC-02 (Reference 19; Appendix C) summarizes the currently-available 28-day compressive strength test results and the buildings associated with those results. NextEra may need to retrieve additional original construction records to implement this approach. [4]

In addition, NextEra has a statistical analysis of over 5,000 compressive strength specimens representing 12 mix classes used during original construction (Reference 20). These data could be applied if NextEra can identify the mix class used for a particular concrete surface.

3.3.2 Approach 2: Reference Cores

An alternative approach for determining the original elastic modulus is to obtain and test reference cores for elastic modulus from concrete at Seabrook Station that is not affected by ASR. The elastic modulus determined using the reference cores would then be applied as equivalent to the 28-day elastic modulus ($E_{o,\text{ ref. core}}$).

NextEra has installed through-thickness expansion monitoring instrumentation in "control" locations where ASR has not affected the concrete. NextEra would test the cores obtained during installation to obtain elastic modulus results.

To implement this approach, NextEra would need to justify that the reference cores were representative of original construction concrete for the location in question. Petrographic examination of the cores (potentially after elastic modulus testing) would conclusively determine that the reference core is not affected by ASR. The original construction data discussed in Appendix C indicate that there are differences in material properties among the buildings at Seabrook Station. NextEra should evaluate selection of a representative reference core on a case-by-case basis.

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[4] Seabrook Station has installed thirty-eight extensometers and has provided MPR with applicable original construction records (if available) and material property information (i.e., elastic modulus data from cores taken at the locations of interest). MPR calculated the expansion-to-date using the data provided by Seabrook and the correlation described in Section 4. The values are recorded in Reference 27; Appendix D.

3.3.3 Selection of an Approach for Determining Original Elastic Modulus

The approach (Approach 1 or Approach 2) should be selected based on specific considerations of the area being evaluated. If both approaches are feasible, both approaches may be used to validate the results using two independent means.

4 Recommended Approach

4.1 Overview of Approach

MPR recommends the following approach for determining ASR-induced through-thickness expansion for instrumented locations at Seabrook Station.

1. Determine the current elastic modulus of the concrete by testing of cores removed from the structure. Elastic modulus testing requires companion compressive strength testing, so MPR recommends obtaining a minimum of four specimens. Two test specimens are for compressive strength testing and two test specimens are for subsequent elastic modulus testing.

2. Establish the original elastic modulus of the concrete by one of the following methods:

   - Using the ACI 318-71 correlation to calculate elastic modulus from 28-day compressive strength test results.

   - Obtaining cores from ASR-free locations and testing for elastic modulus.

3. Calculate the reduction in elastic modulus by finding the ratio of the test result from the ASR-affected area to the original elastic modulus.

4. Quantify through-thickness expansion from original construction to the time the extensometer is installed using the correlation developed in this report. The correlation relates reduction in elastic modulus with measured expansion from beam specimens used during the large-scale ASR structural testing program. A normalized modulus reduction factor of [redacted] discussed in Section 4.2, is used to address uncertainty.

5. Calculate the total expansion by adding the extensometer measurement to the expansion at the time of instrument installation.

4.2 Uncertainty Considerations

This section discusses the sources of uncertainty and summarizes the impact it has on the recommended approach.

4.2.1 Taking Cores

Dimensions

The approximate dimensions of the cores obtained from the large-scale test specimens and the cores obtained from Seabrook Station will be nominally identical (4-inch diameter and 8-inch length). Material tests of specimens with dimensions that are nominally identical will not require corrections for reduced size specimens. Consequently, uncertainty associated with size variation will be negligible for the majority of the cores obtained from Seabrook Station.

MPR acknowledges that, in some cases NextEra may not be able to obtain cores of the planned length due to the fact that the core boring process can result in fracture of the core specimen, which reduces the usable length for material property testing. ASTM guidance regarding reduced length cores will be followed in these situations.

Orientation

The orientation of cores obtained from the large-scale test specimens and cores obtained from Seabrook Station will also be the same (i.e., perpendicular to the embedded reinforcement mats). Therefore, there is no uncertainty related to orientation of the core relative to the dominant direction of expansion.

Location

For the MPR/FSEL test programs, the cores were obtained from locations that were in the central portion of the test specimens, through the openings in the reinforcement mats. These locations were not subject to edge effects and laboratory procedures required strict controls on specimen treatment (e.g., exposure in the environmental conditioning facility). Therefore, exposure conditions were consistent across the entire specimen and variability in ASR development within a test specimen was low.

At Seabrook Station, ASR-related expansion is not typically consistent across a single concrete member. As a result, the locations for extensometer placement (and therefore coring) will be in the areas that have the greatest symptoms of ASR-related expansion. This approach will conservatively characterize the elastic modulus of the concrete member in question.
Because this approach is inherently conservative, quantitative treatment of uncertainty for core location is not necessary.

4.2.2 Methodology

Adjusted Correlation

A normalized modulus reduction factor of [redacted] is applied to Equation 1 to add conservatism to the calculated through-thickness values. This added conservatism helps to address the uncertainty associated with the original modulus (calculated from the original compressive strength using the ACI 318-71 correlation) and the measurement variability in current modulus.

The reduction factor should be applied using Equation 3.

[redacted] [Equation 3]

Equation 1 (purple line), Equation 3 (green line), and the averages of the FSEL data (blue diamonds) are plotted in Figure 4-1. As shown in the graph, Equation 3 bounds or closely approximates all but one of the FSEL data points.

[redacted]

Figure 4-1. Adjusted Correlation

Equation 3 will yield conservative results on a consistent basis. For example, consider a location in which the ratio of the current elastic modulus to the original elastic modulus (i.e., the normalized elastic modulus) is [redacted]. Use of Equation 1 will result in a through-thickness expansion value of [redacted]% while use of Equation 3 will result in a through-thickness expansion value of [redacted]%. In this case, application of Equation 3 will provide a conservatism of [redacted]% (i.e., [redacted]% delta expansion / [redacted]% nominal expansion), increasing the calculated expansion by [redacted] expansion.

Variability in Current Elastic Modulus

The current elastic modulus at Seabrook Station will be determined using cores from the plant. This process is identical to the approach used to determine the elastic modulus from the test data used to develop the correlation. In addition, elastic modulus testing has been and will continue to be performed per ASTM C469-10. The inherent conservatism provided by use of Equation 3 is sufficient to account for any uncertainty associated with the testing method.

Determining Original Elastic Modulus

For the data used to prepare Equation 1, the original elastic modulus is the average elastic modulus test result from cylinders tested 28 days after test specimen fabrication. There are differences between the data used to obtain Equation 1 and the data that will be used to determine pre-instrument expansion at Seabrook Station.

- For Approach 1 at Seabrook Station, the original elastic modulus will be calculated using the 28-day compressive strength test results and the correlation in ACI 318-71.

- For Approach 2 at Seabrook Station, the original elastic modulus will be calculated using the average elastic modulus test result of "reference" cores obtained at the time of extensometer installation. The reference cores will be obtained from a nearby area from the same concrete placement that does not exhibit signs of ASR.

As shown in Figure 3-5, the ACI 318-71 correlation calculates the original elastic modulus within ±20% of the actual original elastic modulus. In instances in which the correlation over-predicts the original elastic modulus, use of the correlation adds conservatism to the approach. In instances in which the correlation under-predicts the original elastic modulus, application of the normalized modulus reduction factor [redacted] adds sufficient conservatism to account for the delta.

It is important to note that Approach 2 uses cores, so the variability associated with the ACI 318-71 correlation is not applicable.

5 Implementation of Recommended Approach

5.1 EXPANSION-TO-DATE AT SEABROOK STATION

Seabrook Station has installed extensometers to monitor through-thickness expansion. Thirty-eight extensometers were installed in ASR-affected locations as of September 2017. Cores were taken at each extensometer location and elastic modulus testing was performed to support determination of the through-thickness expansion-to-date (i.e., the pre-instrument expansion values). The original elastic modulus values were determined using Approach 1. The through-thickness expansion values were calculated using Equation 3, which includes the normalized modulus reduction factor of [redacted].

Determination of the pre-instrument through-thickness expansion values is documented in MPR Calculation 0326-0062-CLC-04 (Reference 27; Appendix D). The results of the calculation are provided in Table 5-1 for reference. NextEra will add these values to the extensometer readings going forward in order to determine the current through-thickness expansion at each extensometer location.

Table 5-1. Through-Thickness Expansion-To-Date (Reference 27; Appendix D)

Location ID Note 1	Through-Thickness Expansion
E1	[redacted]
E2	[redacted]
E3	[redacted]
E4	[redacted]
E5	[redacted]
E6	[redacted]
E7	[redacted]
E8	[redacted]
E9	[redacted]
E10	[redacted]
E11	[redacted]
E12	[redacted]
E13	[redacted] Note 2
E14	[redacted]

Table 5-1. Through-Thickness Expansion-To-Date (Reference 27; Appendix D)

Location ID Note 1	Through-Thickness Expansion
E15	[redacted]
E18	[redacted]
E19	[redacted]
E20	[redacted]
E21	[redacted]
E22	[redacted] Note 2
E23	[redacted] Note 2
E24	[redacted] Note 2
E25	[redacted]
E26	[redacted]
E28	[redacted] Note 2
E29	[redacted]
E30	[redacted]
E31	[redacted]
E32	[redacted]
E33	[redacted]
E35	[redacted] Note 2
E36	[redacted] Note 2
E37	[redacted] Note 2
E39	[redacted]
E40	[redacted]
E41	[redacted] Note 2
E42	[redacted] Note 2
E43	[redacted] Note 2

Notes:

1. Locations E16, E17, E27, E34, and E38 were deleted from the original scope. Thus, extensometers were not installed at these locations.
2. Through-thickness expansion was calculated using one current elastic modulus value rather than averaging multiple current elastic modulus values.

As noted in Table 5-1, field conditions (e.g., cracked cores) and configuration limitations (e.g., embedded steel and conduits, rebar, etc.) limited the number of cores that could be obtained and tested in some locations. In these cases, only one elastic modulus value was obtained.

As stated in Section 4, it is recommended that Seabrook Station obtain at least two elastic modulus test results from each location of interest and average the results to promote greater accuracy. MPR reviewed the reported elastic modulus values and noted the following:

- All single elastic modulus values are within the range of average elastic modulus values from other locations. This observation suggests that the concrete in locations with only one modulus value is in comparable condition to other locations within the plant, which provides assurance that the values are reasonable.

- Of the eleven locations with only one elastic modulus value, nine have calculated nominal through-thickness expansion values that are very low (i.e., [redacted], See Table 5-2). Therefore, the effects of minor inaccuracies associated with the elastic modulus obtained at these locations are insignificant. NextEra is investigating the two locations with higher nominal through-thickness values.

5.2 Conservatism in Through-Thickness Expansion from the Normalized Modulus Reduction Factor

Table 5-2 compares the resultant through-thickness values for the thirty-eight extensometer locations using Equation 1 (i.e., nominal) and Equation 3 (i.e., adjusted) to assess the level of conservatism provided by using a normalized modulus reduction factor of [redacted].

Table 5-2. Comparison of Through-Thicknesses for Equations 1 and 3 (Reference 27; Appendix D)

Location ID Note 1	Nominal Through-Thickness Expansion (Equation 1)	Adjusted Through-Thickness Expansion (Equation 3)
E1	[redacted]	[redacted]
E2	[redacted]	[redacted]
E3	[redacted]	[redacted]
E4	[redacted]	[redacted]
E5	[redacted]	[redacted]
E6	[redacted]	[redacted]
E7	[redacted]	[redacted]
E8	[redacted]	[redacted]
E9	[redacted]	[redacted]
E10	[redacted]	[redacted]
E11	[redacted]	[redacted]
E12	[redacted]	[redacted]
E13 Note 2	[redacted]	[redacted]
E14	[redacted]	[redacted]

Table 5-2. Comparison of Through-Thicknesses for Equations 1 and 3
(Reference 27; Appendix D)

Location ID Note 1	Nominal Through-Thickness Expansion (Equation 1)	Adjusted Through-Thickness Expansion (Equation 3)
E15	[redacted]	[redacted]
E18	[redacted]	[redacted]
E19	[redacted]	[redacted]
E20	[redacted]	[redacted]
E21	[redacted]	[redacted]
E22 Note 2	[redacted]	[redacted]
E23 Note 2	[redacted]	[redacted]
E24 Note 2	[redacted]	[redacted]
E25	[redacted]	[redacted]
E26	[redacted]	[redacted]
E28 Note 2	[redacted]	[redacted]
E29	[redacted]	[redacted]
E30	[redacted]	[redacted]
E31	[redacted]	[redacted]
E32	[redacted]	[redacted]
E33	[redacted]	[redacted]
E35 Note 2	[redacted]	[redacted]
E36 Note 2	[redacted]	[redacted]
E37 Note 2	[redacted]	[redacted]
E39	[redacted]	[redacted]
E40	[redacted]	[redacted]
E41 Note 2	[redacted]	[redacted]
E42 Note 2	[redacted]	[redacted]
E43 Note 2	[redacted]	[redacted]

Notes:

1. Locations E16, E17, E27, E34, and E38 were deleted from the original scope. Thus, extensometers were not installed at these locations.

2. Through-thickness expansion was calculated using one current elastic modulus value rather than averaging multiple current elastic modulus values.

Key observations include the following:

- For the highest through-thickness expansion value of [redacted] (location E21), use of Equation 3 increased the expansion value to [redacted] (i.e., + [redacted] expansion). The impact of the normalized modulus reduction factor (in absolute terms) increases with ASR progression (i.e., at higher levels of expansion).

- In relative terms, application of Equation 3 to the highest through-thickness expansion value (location E9) produced a conservatism of [redacted] (i.e., [redacted] expansion / [redacted] expansion).

Furthermore, the relative conservatism associated with Equation 3 is higher at lower ASR progression levels. As an example, for location E1, where nominal expansion is [redacted], the relative conservatism of using Equation 3 is [redacted] (i.e., [redacted] expansion/ [redacted] expansion).

6 References

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7. Test Program Drawings

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   7.2. FSEL Drawing DWG_0326-0063_[redacted]-G, [redacted]*-Inch Anchorage Specimen Geometry*, Revision 3.

   7.3. FSEL Drawing DWG_0326-0063_[redacted]-R1, [redacted]*-Inch Shear Specimen Reinforcement*, Revision 1.

   7.4. FSEL Drawing DWG_0326-0063_[redacted]-R1, [redacted]*-Inch Anchorage Specimen Reinforcement*, Revision 3.

   7.5. FSEL Drawing DWG_0326-0063_[redacted]-R2, [redacted]*-Inch Instrumentation Specimen Reinforcement - Assembly*, Revision 0.

   7.6 FSEL Drawing DWG_0326-0063_[redacted]-I, [redacted]*-Inch Shear Specimen Instrumentation*, Revision 1.

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A

Correlation Between Expansion and Elastic Modulus

This appendix includes MPR Calculation 0326-0062-CLC-03, *Correlation Between Through-Thickness Expansion and Elastic Modulus in Concrete Test Specimens Affected by Alkali-Silica Reaction (ASR)*, Revision 3.

Non-Proprietary Version

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MPR CALCULATION TITLE PAGE

Client: NextEra Energy Seabrook	Page 1 of 14 + Appendix A and B (32 pages total)
Project: Approach for Estimating Through-Wall Expansion from Alkali-Silica Reaction at Seabrook Station	Task No. 0326-1405-0074
Title: Correlation Between Through-Thickness Expansion and Elastic Modulus in Concrete Test Specimens Affected by Alkali-Silica Reaction (ASR)	Calculation No. 0326-0062-CLC-03

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Michael Saitta February 2, 2015	Vaibhav Bhide February 2, 2015	John W. Simons February 2, 2015	0
Michael Saitta June 23, 2015	Kathleen Mulvaney June 23, 2015	John W. Simons June 23, 2015	1
Amanda E. Card July 19, 2016 (Page 1 to 12+Appendix A)	Keith Means July 19, 2016 (Page 1 to 12 +Appendix A)	John Simons July 19, 2016 (Page 1 to 12 +Appendix A)	2
Keith Means Keith Means July 19, 2016 (Appendix B)	Amanda Card Amanda Card July 19, 2016 (Appendix B)	John W. Simons John W. Simons July 19, 2016 (Appendix B)	2
Amanda Card Amanda E. Card September 6, 2017 (Page 1 to 14+Appendix A)	David Cowles September 6, 2017 (Page 1 to 14 +Appendix A)	John W. Simons John Simons September 6, 2017 (Page 1 to 14 +Appendix A)	3

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the QA requirements of 10CFR50 Appendix B and/or ASME NQA-1, as specified in the MPR Nuclear Quality Assurance Program.

MPR

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RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0326-0062-CLC-03	Amanda Card	[signature]	

Revision	Affected Pages	Description
0	All	Initial Issue
1	All	Added correction factor for through-thickness expansion values to account for influence of mid-plane cracks on the expansion measured using embedded rods.
2	All	Added final test results, updated figures, and revised correlation equation.
3	Page 1 to 14 and Appendix A	Incorporated additional literature data, updated figures, and made minor editorial changes. (Appendix B not revised.)

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

MPR			MPR Associates, Inc. 320 King Street Alexandria, VA 22314
Calculation No. 0326-0062-CLC-03	Prepared By Amanda Card	Checked By [signature]	Page: 3 Revision: 3

Table of Contents

1.0 PURPOSE

This calculation determines a correlation between through-thickness expansion and normalized elastic modulus of concrete test specimens affected by Alkali-Silica Reaction (ASR). The correlation is based on data from test programs that MPR sponsored at Ferguson Structural Engineering Laboratory (FSEL). The correlation is compared to published data.

2.0 SUMMARY OF RESULTS

There is a strong correlation between elastic modulus and through-thickness expansion of concrete specimens that are affected by ASR. The data were fit with a least squares regression using a [redacted] form. Figure 2-1 below shows the FSEL test data and the least squares fit. The least squares fit compares favorably with the trend observed in the data. The R^2 value of the correlation is [redacted]. Figure 2-1 also shows data found in the literature for free expansion of ASR-affected concrete specimens. These data are consistent with the FSEL data.

[redacted]

Figure 2-1. Strong Correlation between Elastic Modulus and Through-Thickness Expansion

3.0 BACKGROUND

Published data show that the material properties of ASR-affected concrete change with increasing levels of ASR-related expansion. The relationship between material properties and

ASR-related expansion will be used to determine the through-thickness expansion of concrete structures at Seabrook Station.

This relationship is defined using data from test programs that MPR sponsored at FSEL (MPR/FSEL test programs) to investigate ASR in reinforced concrete elements. The test specimens were consistent with structures at Seabrook Station in terms of reinforcement details, depth of cover, and overall depth. In addition, the concrete used in the test specimens was representative of the concrete used at Seabrook Station, with some deviations to produce significant ASR-related expansion in a short timeframe.

4.0 ASSUMPTIONS

4.1 Assumptions with a Basis

There are no assumptions with a basis.

4.2 Unverified Assumptions

There are no unverified assumptions.

5.0 DISCUSSION

5.1 Test Data

The test data used herein are for test specimens from the Shear Test Program and the Reinforcement Anchorage Test Program, as well as the Instrumentation Test Program. Combining data from these three programs is appropriate as the same concrete mix was used in all test specimens. In addition, the test specimen configurations and reinforcement details were similar (Reference 6).

Data from all ASR-affected test specimens are used in this calculation. This includes data from [redacted] test specimens: [redacted] reinforcement anchorage specimens, [redacted] shear specimens, and the instrumentation beam.

The baseline material properties are the 28-day tests performed on cylinders molded at the time of concrete placement. The material properties at various levels of ASR-related expansion are based on tests of cores removed from the test specimen prior to structural testing. The data include the following:

- 28 days after concrete placement (before ASR-related expansion occurred)
  - Three compressive strength values,
  - Three elastic modulus values, and

- Three splitting tensile strength values[1].

- Prior to structural testing (after ASR-related expansion occurred)
  - Three compressive strength values,
  - Three elastic modulus values,
  - Three splitting tensile strength values[2], and
  - Through-thickness expansion values.

All values are taken from MPR-4262 (Reference 6) or Reference 8 and are summarized in Appendix A.

5.2 Selection of Elastic Modulus as the Property for the Correlation

To facilitate comparisons, the material properties of each test specimen from the post-ASR cores were normalized against its average value from the 28-day cylinders. Therefore, a sample that had seen very little change in a material property would have a normalized value of approximately 1, whereas one that had experienced a 25% reduction in a material property would have a normalized value of 0.75.

Figure 5-1 plots the normalized compressive strength and the normalized elastic modulus versus through-thickness expansion. From the plot, it appears that there is a strong correlation between modulus and through-thickness expansion. There also appears to be a weak correlation between compressive strength and through-thickness expansion.

There were insufficient data to normalize the splitting tensile strength. Therefore, the splitting tensile strength was plotted against through-thickness expansion in Figure 5-2. There does not appear to be a correlation between splitting tensile strength and expansion. Therefore, it is determined that elastic modulus is the best choice to correlate against expansion.

[1] Note that 28-day results for splitting tensile strength are not available for specimens that were cast before May 2014 ([redacted]) (Reference 6).

[2] Note that the test programs did not start performing splitting tensile testing until the end of May 2014. Therefore, test-day splitting tensile strength test results are not available for [redacted]. Procedure 5-6 allows omission of splitting tensile tests on cores due to the difficulty in extracting testable cores from members with significant cracking due to ASR. Using this provision, splitting tensile strength testing was not performed on cores from [redacted]. Similarly, only two cores from [redacted] and one core from [redacted] were tested (Reference 6).

[redacted]

Figure 5-1. Normalized Compressive Strength and Modulus vs. Through-Thickness Expansion

[redacted]

Figure 5-2. Splitting Tensile Strength vs. Through-Thickness Expansion

5.3 Elastic Modulus Correlation

Non-linear least squares regression was used to fit a curve for the correlation between normalized modulus and expansion. Based on scoping analysis of several types of equations (e.g. natural log, exponential, power, etc.), it was determined that the best-fit curve would take the form of:

[redacted]

Least squares fitting was used to determine the constants A and B. The process of least squares is described in detail in Appendix B. This resulted in a final correlation of:

[redacted]

Where:

expansion is the relative through-thickness expansion of the concrete specimen (0.02 implies a 2% expansion) and
modulus is the normalized modulus of the test specimen after ASR.

This correlation is shown in below in Figure 5-3. The least squares fit compares favorably with the observed data. The R^2 value for the correlation is [redacted].

[redacted]

Figure 5-3. Normalized Modulus vs. Through-Thickness Expansion: Test Data

5.4 Comparison to Published Values

Data on the elastic modulus as a function of ASR-related expansion are available in the literature. These data are for free expansion of small concrete specimens. Table 5-1 lists data from References 3, 4, 5, 10, 11, 12, and 13.

Table 5-1. Existing Data Showing Expansion and Corresponding Elastic Modulus

Expansion (%)	Normalized Elastic Modulus (%)	Reference
0.05	100	3, Table 2.1
0.10	70	3, Table 2.1
0.25	50	3, Table 2.1
0.50	35	3, Table 2.1
1.00	30	3, Table 2.1
1.50	20	3, Table 2.1
0.002	100	4
0.039	66.0	4
0.114	65.2	4
0.210	54.7	4
0.328	50.2	4
0.392	46.7	4
0.007	100	4
0.020	97.7	4
0.038	91.2	4
0.095	78.3	4
0.128	75.8	4
0.29[1]	86.5[2]	5
1.253[1]	13.9[2]	5
0.43[1]	70.2[2]	5
1.573[1]	13.7[2]	5
0.43[1]	39.7[2]	5
1.656[1]	10.3[2]	5
0.43[1]	32.8[2]	5
1.686[1]	8.1[2]	5
0.01	101	10
0.01	101	10
0.04	91.1	10

Table 5-1. Existing Data Showing Expansion and Corresponding Elastic Modulus

Expansion (%)	Normalized Elastic Modulus (%)	Reference
0.08	95.3	10
0.11	93.9	10
0.01	108	10
0.02	89.8	10
0.07	83.6	10
0.12	55.7	10
0.18	57.0	10
0.15	53.4	11
0.18	40.9	11
0.12	104	11
0.15	90.0	11
0.13	82.2	11
0.14	79.0	11
0.01	101	12
0.11	69.6	12
0.18	61.9	12
0.27	51.6	12
0.38	45.3	12
0.42	55.4	12
0.10	103	12
0.05	89.7	12
0.07	85.8	12
0.14	83.4	12
0.08	80.0	12
0.17	72.3	12
0.35	60.5	12
0.08	85.7	12
0.12	82.2	12

Table 5-1. Existing Data Showing Expansion and Corresponding Elastic Modulus

Expansion (%)	Normalized Elastic Modulus (%)	Reference
0.18	74.9	12
0.04	83.8	13
0.04	74.0	13
0.10	64.7	13
0.10	63.5	13

Note 1: Longitudinal prism expansion was selected as the most representative.

Note 2: Taken as elastic modulus at testing divided by elastic modulus at 28 days.

Figure 5-4 plots these data and compares them to the FSEL data and to the correlation based on the FSEL data. The data from published literature follow a trend that is consistent with the FSEL test data and the correlation determined using these data.

[redacted]

Figure 5-4. Normalized Modulus vs. Through-Thickness Expansion: Published Literature

6.0 REFERENCES

1. Not Used.

2. Bayrak, Oguzhan, *Structural Implications of ASR: State of the Art*, July 28, 2014, transmitted to Seabrook Station in MPR Letter 0326-0058-200, dated July 29, 2014.

3. Clark, L.A., *Critical Review of the Structural Implications of the Alkali Silica Reaction in Concrete*, Transport and Road Research Laboratory Contractor Report 169, July 1989.

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5. Ahmed, T. et al., *The effect of Alkali Reactivity on the Mechanical Properties of Concrete*, Construction and Building Materials, 17 (2003) 123-144, January 9, 2002.

6. MPR-4262, "Shear and Reinforcement Anchorage Testing of Concrete Affected by Alkali-Silica Reaction," Volume I, Revision 1 & Volume II, Revision 0. (Seabrook FP#100994)

7. MPR-4259, "Commercial Grade Dedication Report for Seabrook ASR Shear, Reinforcement Anchorage and Instrumentation Testing," Revision 0. (Seabrook FP # 100995)

8. Special Test and Inspection Reports (STIRs) as accepted by CGAR-0326-0062-43-2 Revision 0, CGAR-0326-0062-43-5 Revision 1, and CGAR-0326-0062-43-7 Revision 0.

   a) STIR-0326-24-103

   b) STIR-0326-24-104

   c) STIR-0326-24-105

   d) STIR-0326-24-147

   e) STIR-0326-24-204

   f) STIR-0326-24-228

9. MPR-4286, "Supplemental Commercial Grade Dedication Report for Seabrook Test Programs," Revision 0. (Seabrook FP# 101003)

10. Espisito, R. et al, *Influence of the Alkali-Silica Reaction on the Mechanical Degradation of Concrete,* Journal of Materials in Civil Engineering, Vol. 28, No. 6, Article No. 04016007, June 2016.

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Calculation No.	Prepared By	Checked By	Page: A-1
0326-0062-CLC-03	Amanda Card	[signature]	Revision: 3

A
Test Data

This Appendix includes tables of summarized test data originally from FSEL. Table A-1 contains data from tests conducted 28 days after casting. The data are used to normalize the post-ASR data. Table A-2 contains data from tests that were conducted after ASR had occurred (i.e., post-ASR data). Table A-3 contains the through-thickness expansion values. Test data are taken from Reference 6 or the main body of this calculation unless otherwise noted. Applicable Special Test Inspection Records (STIRs) are listed for reference.

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Calculation No.	Prepared By	Checked By	Page: A-2
0326-0062-CLC-03	Amanda Card	[signature]	Revision: 3

Table A-1. FSEL 28-Day Compressive Strength, Elastic Modulus, and Splitting Tensile Strength Test Data

[redacted]

Table A-1. FSEL 28-Day Compressive Strength, Elastic Modulus, and Splitting Tensile Strength Test Data

[redacted]

Table A-1. FSEL 28-Day Compressive Strength, Elastic Modulus, and Splitting Tensile Strength Test Data

[redacted]

Table A-2. FSEL Average Expansion, Compressive Strength, and Elastic Modulus: Test Data After ASR

[redacted]

Table A-2. FSEL Average Expansion, Compressive Strength, and Elastic Modulus: Test Data After ASR

[redacted]

Table A-2. FSEL Average Expansion, Compressive Strength, and Elastic Modulus: Test Data After ASR

[redacted]

Table A-3. FSEL Expansion Test Data With Correction Factor

[redacted]

B
Least Squares Regression

Purpose

This appendix explains the methodology used to perform the Least Squares Regression Analysis. A brief description of the fit statistic R^2 is also given. After the method of Least Squares is explained, the method is applied to the correlation between the FSEL test data for normalized elastic modulus and corrected through thickness expansion.

Discussion

Least Squares Regression is a commonly accepted method of fitting a curve to a set of scattered data. This is done by minimizing the sum of squares error term. This is a common statistical method that is documented in textbooks such as "Applied Data Analysis and Modeling for Energy Engineers and Scientists" by T.A. Reddy. The sum of squares is given by:

$$S = \sum_{i=1}^{m} r_i^2$$

Where:

S is the error term,
m is the number of known values, and
r_i is the residual of the ith value, as given by:

$$r_i = y_i - f(x_i, C)$$

Where:

y_i and x_i are a known value pair,
f is the regressed or fit function, and
C is the set of constants used to fit the model.

By combining the above equations with a known set of values, S is minimized by varying C. In some cases, this can be accomplished analytically, but is often accomplished numerically. The values of C that minimize S are said to be the fitting parameters, and the function $f(x_i, C)$ is the curve of best fit in the least squares sense.

It is often desirable to determine how well a given curve fits a set of data. A commonly used statistic to determine this is the coefficient of determination, R^2. R^2 is defined as:

$$R^2 = 1 - \frac{SS_{res}}{SS_{tot}}$$

$$SS_{res} = \sum_{i=1}^{m} (y_i - f(x_i, C))^2 = S$$

$$SS_{tot} = \sum_{i=1}^{m} (y_i - \bar{y})^2$$

Calculation

The least squares regression performed in the main body of this calculation is described in detail below. The set of points is listed in Table B-1 and plotted in Figure B-1.

Table B-1. Known Values

[redacted]

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0326-0062-CLC-03	[redacted]	Amanda Card	Revision: 2

Table B-1. Known Values

[redacted]

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Calculation No.	Prepared By	Checked By	Page: B-4
0326-0062-CLC-03	[redacted]	Amanda Card	Revision: 2

Table B-1. Known Values

[redacted]

[redacted]

Figure B-1. Plot of Known Values

It appears that a natural log fit is reasonable. Therefore, it can be fit to an equation of form:

[redacted]

Where:

x is the set of values of X as shown in Table B-1.
A and B are a set of constants (C) used to fit the model.

To begin, we will guess at the values of A and B. In this example, our first guess will be that $A = -0.1$ and $B = -0.5$. Using the model given above, we compute a value for y at each given x. For each computed value, the residual is also computed. These values are shown in Table B-2.

Table B-2. Example Values With Computed Residuals

[redacted]

Table B-2. Example Values With Computed Residuals

[redacted]

Table B-2. Example Values With Computed Residuals

[redacted]

Taking the sum of squares of the residuals, we find a value of approximately [redacted]. However, this can be improved on. To do so, we iteratively adjust the values A and B to minimize S.

Values of $A =$ [redacted] and $B =$ [redacted] result in S being minimal and provide a good estimate of the solution. The fitted curve is plotted against the data in Figure B-2. The newly computed values are shown in Table B-3. The regressed equation is:

[redacted]

Table B-3. Example Values With Computed Residuals - Updated

[redacted]

Table B-3. Example Values With Computed Residuals - Updated

[redacted]

Table B-3. Example Values With Computed Residuals - Updated

[redacted]

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Calculation No.	Prepared By	Checked By	Page: B-10
0326-0062-CLC-03	[redacted]	Amanda Card	Revision: 2

[redacted]

Figure B-2. Regressed Curve

R^2 can now be computed using the regressed curve. The sum of squared residuals is [redacted] (SS_{res}). The mean of y is [redacted]. Therefore, the sum of squared totals is [redacted] (SS_{tot}). R^2 can now be computed.

$$R^2 = 1 - SS_{res}/SS_{tot}$$

$$R^2 = \text{[redacted]}$$

B
Evaluation of ACI Equation for Elastic Modulus

This appendix includes MPR Calculation 0326-0062-CLC-01, *Evaluation of ACI Equation for Elastic Modulus*, Revision 0.

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CALCULATION TITLE PAGE

Client: NextEra Energy Seabrook, LLC	Page 1 of 12+ Appendix A and B
Project: Approach for Estimating Through-Wall Expansion from Alkali-Silica Reaction at Seabrook Station	Task No. 0326-1405-0074
Title: Evaluation of ACI Equation for Elastic Modulus	Calculation No. 0326-0062-CLC-01

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
Amanda Card Amanda Card 01/29/2015	David H. Bergquist DHBerg 01/29/2015	John W. Simons John W. Simons 01/29/2015	0

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the QA requirements of 10CFR50 Appendix B and/or ASME NQA-1, as specified in the MPR Nuclear Quality Assurance Program.

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RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0326-0062-CLC-01	Amanda Card	[signature]	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

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Calculation No.	Prepared By	Checked By	Page: 3
0326-0062-CLC-01	Amanda Card	[signature]	Revision: 0

Table of Contents

1.0 INTRODUCTION

1.1 Purpose

This calculation evaluates the applicability of the elastic modulus equation provided in Section 8.5.1 of ACI 318-71 (Reference 2) to the concrete mix used in the Beam Test Programs that MPR is sponsoring at Ferguson Structural Engineering Laboratory (FSEL).

1.2 Background

MPR is developing a methodology to determine the through-thickness expansion of concrete structures at Seabrook Station due to Alkali-Silica Reaction (ASR). The through-thickness expansion results in a reduction in the elastic modulus. One approach for estimating the original elastic modulus (i.e., the elastic modulus before ASR expansion occurs) is to calculate it using the 28-day compressive strength of the concrete and the equation provided in ACI 318-71.

2.0 SUMMARY OF RESULTS AND CONCLUSIONS

Based on the results of this calculation, the relationship between the measured 28-day compressive strength and the elastic modulus for the test specimens within the Beam Test Programs at FSEL is consistent with the ACI equation. The measured data and calculated results show a similar trend. Measured and calculated elastic modulus values for all but three data sets were within the variability range stated in Reference 2, 20%.

3.0 APPROACH

Section 8.5.1 of ACI 318-71 (Reference 2) states that the 28-day elastic modulus (E_c) of concrete can be calculated based on the density of concrete in lb/ft^3 (w_c) and the 28-day compressive strength of concrete (f_c'). This relationship is expressed using Equation 1.

$$E_c = 33 w_c^{1.5} \sqrt{f_c'} \qquad (1)$$

Section R8.5.1 of ACI 318 (Reference 2) also states that measured values for elastic modulus range from 80% to 120% of the calculated value.

Reference 3 provides the basis for Equation 1 and supports Reference 2. Equation 1 is based on light weight and normal weight concrete test data from various published articles and unpublished reports from the Expanded Shale, Clay, and Slate Institute.

The elastic modulus for normal weight concrete (approximate density of $144 \frac{\text{lb}}{\text{ft}^3}$) can be calculated using Equation 2, a simplified version of Equation 1. (Reference 2)

$$E_c = 57{,}000\sqrt{f_c'} \qquad (2)$$

As part of the Shear and Reinforcement Anchorage Test Programs and Instrumentation Specimen Testing, FSEL has determined the 28-day concrete elastic modulus and compressive strength for each beam specimen fabricated to date. These tests use cylinders molded at the time of concrete placement. In addition to the 28-day data, data are also available from cores removed from the test specimens used for control tests (i.e., tests performed shortly after 28 days, before the onset of deleterious ASR expansion). The results of the FSEL elastic modulus and compressive strength tests are compared to Equation 2 (and therefore Equation 1) in this calculation to confirm that the ACI equation is applicable to the concrete mix used in the Beam Test Programs.

4.0 INPUTS

As stated in Section 3.0, the 28-day elastic modulus and the 28-day compressive strength of twenty beams, collected by FSEL, were used to confirm the applicability of Equations 1 and 2. A total of [redacted] data sets were evaluated.

The data were taken from the Special Test and Inspection Records (STIRs) listed in Table 1. (Reference 5 through Reference 40)

Table 1. References for Test Data

[redacted]

Table 1. References for Test Data

[redacted]

5.0 CALCULATION

5.1 Concrete Density Verification

It is important to note that the density of concrete varies slightly among the beams that were tested. However, all test beams are composed of normal weight concrete ($144\frac{\text{lb}}{\text{ft}^3}$).

The simplified equation for normal weight concrete, Equation 2, is therefore applicable and was used to calculate the elastic moduli reported in this calculation.

The relevance of Equation 2 was verified by calculating the density of a beam and comparing it to the density of normal weight concrete. The two values agreed.

A sample density calculation is provided in Appendix A.

5.2 Elastic Modulus Determination

The average 28-day compressive strengths and Equation 2 were used to calculate the 28-day elastic modulus for each of the [redacted] data sets listed in Table 1. The percent error is calculated between the measured and calculated elastic modulus values.

The calculation is provided in Appendix B.

6.0 RESULTS AND CONCLUSIONS

The measured elastic modulus values for the [redacted] data sets collected at FSEL align well with the calculated elastic modulus values (from Equation 2). All but [redacted] of the measured elastic modulus values are within 80% to 120% of the calculated value.

Figure 1 compares the FSEL data to the trendline for Equation 2.

Figure 2 and Figure 3 illustrate that nearly all of the FSEL data falls within 80% and 120% of the calculated elastic modulus value, which is consistent with the statement in Section R8.5.1 of ACI 318 (Reference 2) regarding the accuracy of the equation.

It is important to note that the measured elastic modulus is plotted and compared to the trendline associated with Equation 2 in Figure 1 and Figure 2. The percent difference between measured elastic modulus and calculated elastic modulus (per Equation 2) is plotted in Figure 3. All three figures support the conclusion that Equation 2 (and therefore Equation 1) applies to the FSEL data.

The calculations required to generate Figure 1, Figure 2, and Figure 3 are also provided in Appendix B. Cylinders are depicted in blue. Cores are depicted in green.

Based on the results of this calculation, the elastic modulus equation, provided in Section 8.5.1 of ACI 318-71, is validated.

[redacted]

Figure 1. Comparison of FSEL Elastic Modulus Test Data with Equation 2

[redacted]

Figure 2. Range of FSEL Elastic Modulus Test Data

[redacted]

Figure 3. Percent Error: FSEL Elastic Modulus Test Data vs. Equation 2 Elastic Modulus

Calculation No.	Prepared By	Checked By	Page: 11
0326-0062-CLC-01	Amanda Card	DGBor	Revision: 0

7.0 REFERENCES

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2. ACI 318-71, "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, 1971.
3. Pauw, A., "Static Modulus of Elasticity of Concrete as Affected by Density," *Journal of the American Concrete Institute*, Vol. 32, No. 6, December 1960, pg. 679-687.
4. United Engineers Calculation No. CD-20, "Design of Mats at El. 20' 0" and 0' 0" and Walls Below Grade for Electrical Tunnels and Control Building," Revision 2.
5. MPR Special Test and Inspection Record No. STIR-0326-0062-24-9, Revision 0.
6. MPR Special Test and Inspection Record No. STIR-0326-0062-24-17, Revision 0.
7. MPR Special Test and Inspection Record No. STIR-0326-0062-24-21, Revision 0.
8. MPR Special Test and Inspection Record No. STIR-0326-0062-24-24, Revision 0.
9. MPR Special Test and Inspection Record No. STIR-0326-0062-24-30, Revision 0.
10. MPR Special Test and Inspection Record No. STIR-0326-0062-24-34, Revision 0.
11. MPR Special Test and Inspection Record No. STIR-0326-0062-24-50, Revision 0.
12. MPR Special Test and Inspection Record No. STIR-0326-0062-24-45, Revision 0.
13. MPR Special Test and Inspection Record No. STIR-0326-0062-24-93, Revision 0.
14. MPR Special Test and Inspection Record No. STIR-0326-0062-24-110, Revision 0.
15. MPR Special Test and Inspection Record No. STIR-0326-0062-24-86, Revision 0.
16. MPR Special Test and Inspection Record No. STIR-0326-0062-24-96, Revision 0.
17. MPR Special Test and Inspection Record No. STIR-0326-0062-24-13, Revision 0.
18. MPR Special Test and Inspection Record No. STIR-0326-0062-24-19, Revision 0.
19. MPR Special Test and Inspection Record No. STIR-0326-0062-24-23, Revision 0.
20. MPR Special Test and Inspection Record No. STIR-0326-0062-24-26, Revision 0.

21. MPR Special Test and Inspection Record No. STIR-0326-0062-24-31, Revision 0.

22. MPR Special Test and Inspection Record No. STIR-0326-0062-24-35, Revision 0.

23. MPR Special Test and Inspection Record No. STIR-0326-0062-24-84, Revision 0.

24. MPR Special Test and Inspection Record No. STIR-0326-0062-24-106, Revision 0.

25. MPR Special Test and Inspection Record No. STIR-0326-0062-24-117, Revision 0.

26. MPR Special Test and Inspection Record No. STIR-0326-0062-24-11, Revision 0.

27. MPR Special Test and Inspection Record No. STIR-0326-0062-24-47, Revision 0.

28. MPR Special Test and Inspection Record No. STIR-0326-0062-24-95, Revision 0.

29. MPR Special Test and Inspection Record No. STIR-0326-0062-24-111, Revision 0.

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32. MPR Special Test and Inspection Record No. STIR-0326-0062-24-87, Revision 0.

33. MPR Special Test and Inspection Record No. STIR-0326-0062-24-107, Revision 0.

34. MPR Special Test and Inspection Record No. STIR-0326-0062-24-90, Revision 0.

35. MPR Special Test and Inspection Record No. STIR-0326-0062-24-123, Revision 0.

36. MPR Special Test and Inspection Record No. STIR-0326-0062-24-124, Revision 0.

37. MPR Special Test and Inspection Record No. STIR-0326-0062-24-127, Revision 0.

38. MPR Special Test and Inspection Record No. STIR-0326-0062-24-128, Revision 0.

39. MPR Special Test and Inspection Record No. STIR-0326-0062-24-135, Revision 0.

40. MPR Special Test and Inspection Record No. STIR-0326-0062-24-136, Revision 0.

A

Sample Concrete Density Calculation

The density of [redacted] was calculated using data provided in STIR-24-90. (Reference 34)

The relevant data and density calculation are provided in Table A-1.

Table A-1. Concrete Density Calculation

[redacted]

B

Test Data and Calculations

The information used to perform this calculation and to generate the graphs included herein is provided in Table B-1and Table B-2.

Table B-1. Compressive Strength and Calculated Elastic Modulus

[redacted]

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Calculation No.	Prepared By	Checked By	Page: B-2
0326-0062-CLC-01	Amanda Card	DGBor	Revision: 0

Table B-1. Compressive Strength and Calculated Elastic Modulus

[redacted]

Calculation No.	Prepared By	Checked By	Page: B-3
0326-0062-CLC-01	Amanda Card	DGBor	Revision: 0

Table B-1. Compressive Strength and Calculated Elastic Modulus

[redacted]

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Calculation No.	Prepared By	Checked By	Page: B-4
0326-0062-CLC-01	Amanda Card	DABor	Revision: 0

Table B-1. Compressive Strength and Calculated Elastic Modulus

[redacted]

Table B-2. Elastic Modulus

[redacted]

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Calculation No.	Prepared By	Checked By	Page: B-6
0326-0062-CLC-01	Amanda Card	DGBor	Revision: 0

Table B-2. Elastic Modulus

[redacted]

Table B-2. Elastic Modulus

[redacted]

C

Compressive Strength of Concrete at Seabrook Station

This appendix includes MPR Calculation 0326-0062-CLC-02, *Compressive Strength Values for Concrete at Seabrook Station*, Revision 0.

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CALCULATION TITLE PAGE

Client: Nex Era Energy S abrook, LLC	Page 1 of 8 plus Appendix A
Project: Appr ach fo Estima ing Through-Wall Expansion from Alkali-Silica Reaction at Seabrook Station	Task No. 0326-1405-0074
Title: Compressive Strength Va u s for Concrete at Seabrook Station	Calculation No. 0326-0074-CLC-02

Preparer / Date	Checker / Date	Reviewer & App over / Date	Rev. No.
David H. Bergquist January 28, 2015	Christina Hamm January 28, 2015	Jo n W. Simons January 28, 015	0

QUALITY ASSURANCE DOCUMENT

This documen has been prepared check d, and reviewed/app o ed n acc ance with the QA requirements of 10CFR50 Appendix B and/or ASME NQA- , as s ecified in h MPR Nuclear Quality Assurance Program.

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RECORD OF REVISIONS

Calculation No.	Prepared By	Che ed By	Page: 2
0326-0074-CLC-02			

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revisi n number found on each individual p ge of the calc ılation carries the revision level f the c lcul tio n effect at the time that page was last revised.*

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Calculation No.	Prepared By	Ch cke ›y	Page: 3
0326-0074-CLC-02			Revision: 0

Table of Contents

1.0 Purpose

This calculation evaluates available 28-day compressive strength values determined from concrete cylinders during the original construction of Seabrook Station. These values are then disp a ed on a h stogram o show the data distribution, m an, nd standard deviation. Additionally, the data are separated by location and by the strengt as of the concrete (i.e. sp ci ed c mpress v strength).

2.0 Summary of Results

All available 28-da ompr ssi e strength da a points were compiled to form the histogram giv n in igure 1. The average 28-day compressive strength is 5456 p and the standard deviation is 568 psi. Seventy-five percent of t e data fall wit i ne stan ard deviation of the mean and ninety-four percent of the dat fall w thin two st nda eviations of the mean.

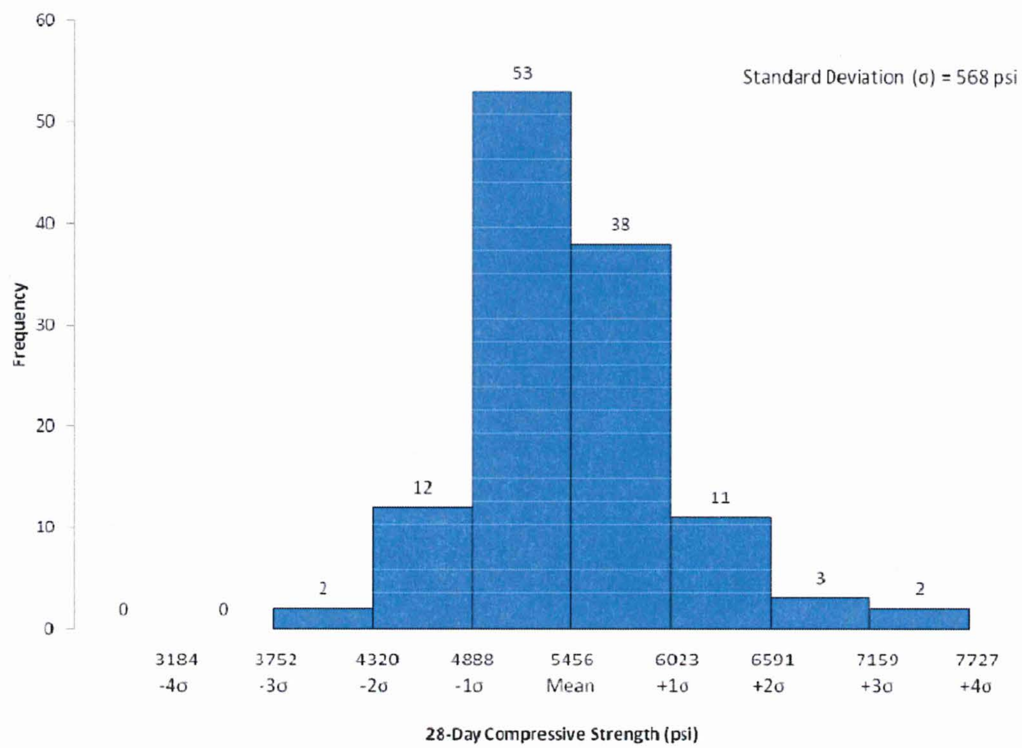

Figure 1. 28-Day Compressive Strength Values for Concrete y i der at Seabrook Station

Table 1 sho the data p esented in Figure 1 along with the data categor zed by room at Seabrook and by con ret strength class.

Table 1. 28-Day ompres ive S ren th Data or Se r ok Station

	Mean	Standard Deviation (σ)	No. Of Data Points	Min	Max	% of data within 1 σ	% of data within 2 σ
All Data	5456	568	121	4240	7360	75%	94%
3000 PSI Strength Class	5621	691	50	4270	7360	74%	96%
4000 PSI Strength Class (Note 1)	5339	430	71	4240	6150	70%	99%
Containment Enclosure Building	5426	380	24	4880	6080	67%	100%
RHR Equipment Vault	5503	491	35	4240	6150	63%	97%
EFW Pump House Stairway A	5390	269	12	4950	5870	67%	100%
RCA Walkway	4891	404	12	4270	5450	50%	100%
B EDG Building	5197	371	21	4600	5840	62%	100%
B Electrical Tunnel	6163	705	17	5220	7360	65%	100%

Note 1: The stren th class of 9 samples f om the RHR Equipm nt Room cannot i ent f ed with certainty due to poor resolu i n of he r ference document. These s mp es are most likely 4000 psi st e h cla samples based on their proximity to other 4000 psi strength class samples. See Appendix A for more details.

3.0 Background

MPR is developing a meth dology to determine the through-thickness ex ansion of concrete structures at Sea roo tation due to the Alkali- il c React o SR he through-thickness expans on is related to t e r ductio n elastic modul s f the concrete over time. One approach for estimating the original l stic modulus is to calculate it from the 28-day compressive strength of the concrete using an equation from ACI 318 (Reference 1).

4.0 METHODOLOGY

S abrook For i n Print No. 100629 and United Engineers Cal ulati n No. CD-20 (References 2 and 3) include 28- ay compressive stren t e ult for co crete sed in original construction for the following build ngs at Seabrook Station:

- Containment Enclosure Building
- RHR Equip nt Vault
- EFW Pump House Stairway A
- RCA Walkway
- B Diesel Gene a or Building
- B Electri al Tun el

These references provide the 121 data points used in this calculation. The se 28-day compressive strength data oints are ncluded in Appendix A.

5.0 RESULTS

The average 28-day co ressive strength of all data points is 5456 si a d the standard deviation is 568 psi. Seventy-five percent of the da a fall ithin one andard deviation of the mean and ninety-four percent o the da a fall w thin two st ndar eviations o the mean. Therefore, the mean is a represe tat ve alue for the 28-day com ressive str ng h of all oncrete used at Seabrook. See Sec ion 2.0 for a histogram of all data points as well as a t ble of the compressive strength data y room and concrete strength class. Fig res nd 3 d sp a the data for the 3000 ps and 4000 psi strength class concrete cores, respectively.

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Calculation No.	Prepared By	Ch cke ;y	Page: 7
0326-0074-CLC-02			Revision: 0

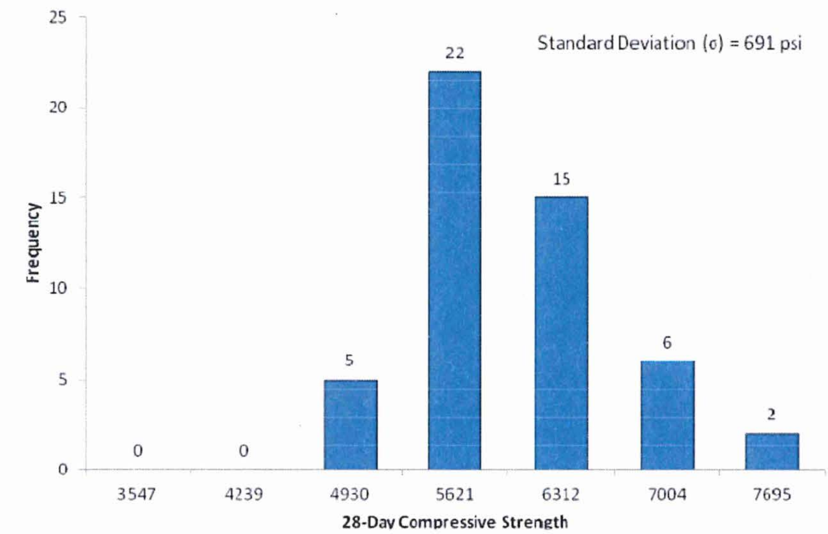

Figure 2. 28-Day Compressive Strength Values f r 300 psi Str ng h Class Concrete Cores

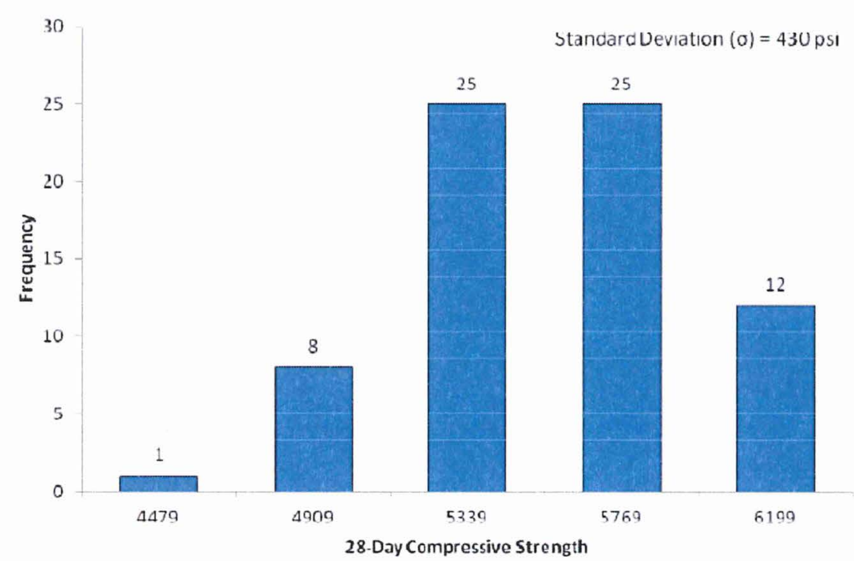

Figure 3. 28-D y C mpre si e Strength Valu s f r 000 si Str ngth Class Concrete Cores

6.0 REFERENCES

1. ACI 318-71, "Bu ldin Co e R quireme ts fo S ru t ral Co crete," American Concrete Institute, 1971.

2. S abro k For i n Print No. 100629, "Conc e e Test e ort." Rev sion 0.

3. United Engineers Calculation No. CD-20, "Design of Ma s at l. 2)' 0" and 0' 0" and Wal s Below G de for Electrical Tunnels and Control Building," Revision 4.

A
Compressive Strength Data

Table A-1 contains the 28-day c mpress v stren th data f r concrete cores at Seabrook Station.

Table A-1: 28-Da o pres iv trengths for Concrete C res at Seabrook Station

Room	Sample No.	Compressive Strength (psi)	S:rength Class (psi)
Containment Enclo ure Building (Reference 2)	4405	5130	4000
	4406	5200	4000
	4407	5620	4000
	4405A	6080	4000
	4406A	5700	4000
	4407A	5410	4000
	4641	5200	4000
	4642	5060	4000
	4643	5410	4000
	4641A	5980	4000
	4642A	6050	4000
	4643A	6010	4000
	4648	5020	4000
	4649	5090	4000
	4650	4950	4000
	4655	5380	4000
	4656	5240	4000
	4657	4880	4000
	4648A	5020	4000
	4649A	5160	4000
	4650A	5360	4000
	4655A	5780	4000
	4656A	5730	4000
	4657A	5770	4000

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Calculation No.	Prepared By	Ch cke ;y	Page: A-2
0326-0074-CLC-02			Revision: 0

Table A-1: 28-Da o pres iv trengths for Concrete C res at Seabrook Station

Room	Sample No.	Compressive Strength (psi)	S:rength Class (psi)
RHR Equi ment Vault (Reference 2)	94	6070	3000
	95	5780	3000
	96	5710	3000
	101	5800	3000
	102	5730	3000
	103	5700	3000
	108	6140	3000
	109	5960	3000
	110	6030	3000
	430	5020	4000[1]
	431	4990	4000[1]
	432	5060	4000[1]
	430A	5450	4000
	431A	5480	4000
	432A	5380	4000
	437	6010	4000
	438	5620	4000
	439	5980	4000
	437A	6010	4000
	438A	6150	4000
	439A	6120	4000
	unknown	4670	4000
	unknown	4740	4000
	unknown	5660	4000
	unknown	5450	4000
	unknown	5480	4000
	unknown	5620	4000
	unknown	5700	4000
	unknown	5700	4000
	unknown	4600	4000[1]
	unknown	5130	4000[1]
	unknown	4240	4000[1]
	unknown	5270	4000[1]
	unknown	5240	4000[1]

[1] Concret st ength class ca not be determined with certainty du to p r res lu ion reference document.

Table A-1: 28-Da o pres iv trengths for Concrete C res at Seabrook Station

Room	Sample No.	Compressive Strength (psi)	S:rength Class (psi)
RHR Equi ment Vault	unknown	4920	4000[1]
EF Pu p House Stairway A (Reference 2)	590	5700	3000
	591	5700	3000
	592	5590	3000
	590A	4950	3000
	591A	5200	3000
	592A	5240	3000
	597A	5290	3000
	598A	5870	3000
	599A	5380	3000
	604A	5180	3000
	605A	5340	3000
	606A	5240	3000
RCA Walkway (Reference 2)	489	5310	3000
	490	4440	3000
	491	4950	3000
	489A	5200	3000
	490A	5450	3000
	491A	4880	3000
	484	4470	3000
	485	4270	3000
	486	4370	3000
	484A	5040	3000
	485A	5090	3000
	486A	5220	3000
B EDG Building (Reference 2)	unknown	4620	4000
	unknown	4700	4000
	unknown	4600	4000
	unknown	5150	4000
	unknown	5660	4000
	unknown	5200	4000
	315	5520	4000
	316	5590	4000
	317	5470	4000
	315A	5840	4000

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Calculation No.	Prepared By	Ch cke ;y	Page: A-4
0326-0074-CLC-02			Revision: 0

Table A-1: 28-Da o pres iv trengths for Concrete C res at Seabrook Station

Room	Sample No.	Compressive Strength (psi)	S:rength Class (psi)
B EDG Building (Reference 2)	316A	5110	4000
	317A	5640	4000
	unknown	4600	4000
	unknown	4950	4000
	unknown	4950	4000
	unknown	5380	4000
	unknown	5310	4000
	unknown	5040	4000
	unknown	5340	4000
	unknown	5040	4000
	unknown	5430	4000
B Electrical Tunnel (Reference 3)	427	5410	3000
	428	5220	3000
	426A	6560	3000
	427A	6490	3000
	428A	6100	3000
	433	5470	3000
	434	5550	3000
	435	5890	3000
	433A	7000	3000
	434A	7220	3000
	435A	7360	3000
	440	5730	3000
	441	5480	3000
	442	5390	3000
	440A	6330	3000
	441A	6810	3000
	442A	6760	3000

D

Through-Wall Expansion from Alkali-Silica Reaction To-Date for Extensometers Installed at Seabrook Station Prior to September 2017

This appendix includes MPR Calculation 0326-0062-CLC-04, *Calculation of Through-Wall Expansion from Alkali-Silica Reaction To-Date Station for Extensometers Installed at Seabrook Station Prior to September 2017*, Revision 1.

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CALCULATION TITLE PAGE

Client: NextEra Energy Seabrook	Page 1 of 15
Project: Approach for Estimating Through-Wall Expansion from Alkali-Silica Reaction at Seabrook Station	Task No. 0326-1405-0074
Title: Calculation of Through-Wall Expansion from Alkali-Silica Reaction To-Date for Extensometers Installed at Seabrook Station Prior to September 2017	Calculation No. 0326-0062-CLC-04

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
Amanda Card Amanda E. Card July 19, 2016	David Cowles July 19, 2016	C W Bagley Christopher Bagley July 19, 2016	0
Amanda Card Amanda E. Card September 7, 2017	David Cowles September 7, 2017	C W Bagley Christopher Bagley September 7, 2017 (Reviewer) John W. Simons John W. Simons September 7, 2017 (Approver)	1

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the QA requirements of 10CFR50 Appendix B and/or ASME NQA-1, as specified in the MPR Nuclear Quality Assurance Program.

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RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0326-0062-CLC-04	Amanda Card	[signature]	

Revision	Affected Pages	Description
0	All	Initial Issue
1	All	Updated to include expansion information from additional extensometer locations at Seabrook Station. Also made minor editorial changes throughout the body of the calculation.

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

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Checked By: [signature]

Table of Contents

1.0 Purpose and Background

This calculation determines the through-thickness expansion to-date from Alkali-Silica Reaction (ASR) for various locations in reinforced concrete structures at Seabrook Station. The current through-thickness expansion values were calculated using a correlation between through-thickness expansion and elastic modulus of concrete test specimens affected by ASR.

Seabrook Station has installed instruments (i.e., extensometers) to monitor through-thickness expansion. This calculation determines the current through-thickness expansion values for each of the installed extensometer locations.

Seabrook Station will follow the process presented in this calculation to determine the current through-thickness expansion values upon installation of extensometers in the future.

2.0 Summary of Results and Conclusion

The table below provides through-thickness expansion values to-date for reinforced concrete locations of interest at Seabrook Station.

Table 2-1. Through-Thickness Expansion To-Date

[redacted]

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Table 2-1. Through-Thickness Expansion To-Date

[redacted]

Table 2-1. Through-Thickness Expansion To-Date

[redacted]

3.0 METHODOLOGY

This calculation uses the equation developed in Reference 3 to determine the current through-thickness expansion from ASR. The equation in Reference 3 uses normalized elastic modulus (i.e., current elastic modulus / original elastic modulus) to determine through-thickness expansion to-date. The key steps in the methodology used herein are (1) determination of the original elastic modulus which was not directly measured during original construction and (2) determination of through-thickness expansion using the equation in Reference 3.
The ASR-affected elastic modulus is determined using measurements of cores removed from the plant structures in the vicinity of the extensometer locations.

3.1 Using 28-Day Compressive Strength to Determine Original Elastic Modulus

Section 8.5.1 of ACI 318-71 (Reference 2) states that the 28-day elastic modulus (E_c) of concrete can be calculated based on the density of concrete in lb/ft^3 (w_c) and the 28-day compressive strength of concrete (f_c'). The elastic modulus for normal weight concrete (approximate density of $144\frac{\text{lb}}{\text{ft}^3}$) can be calculated using Equation 1. Equation 1 was developed using data from a wide range of concrete and is therefore generally applicable to most concrete mixes.

$$E_c = 57{,}000\sqrt{f_c'} \qquad \text{(Equation 1)}$$

Reference 1 evaluates the applicability of Equation 1 to the concrete mix used in the test programs that MPR sponsored at Ferguson Structural Engineering Laboratory (FSEL) (i.e., the MPR/FSEL test programs). Based on the results of Reference 1, the relationship between the measured 28-day compressive strength (original compressive strength) and the 28-day elastic modulus for the test specimens within the MPR/FSEL test programs is consistent with the ACI equation.

Using Equation 1 to evaluate concrete at Seabrook Station is also appropriate. The correlation was demonstrated to apply to the concrete used in the MPR/FSEL test programs in Reference 1 and the concrete mix used in the MPR/FSEL test programs was representative of the concrete at Seabrook Station. Accordingly, the compressive strength of concrete identified in Seabrook Station's original construction records can be used to determine the original elastic modulus (E_c) of the concrete of interest.

3.2 Determining Through Thickness Expansion from Elastic Modulus

Reference 3 determines a correlation (Equation 2) between through-thickness expansion and normalized elastic modulus of concrete test specimens affected by ASR. The correlation is based on data from test programs that MPR sponsored at FSEL. The correlation was verified against published data.

[redacted] (Equation 2)

Where:

expansion is the relative through-thickness expansion of the concrete specimen (e.g., 0.02 equals a 2% expansion) and
modulus is the normalized modulus of the test specimen after ASR.

A normalized modulus reduction factor of [redacted] was applied to Equation 2 to provide appropriate conservatism for the methodology.

4.0 Assumptions

4.1 Verified Assumptions

There are no verified assumptions.

4.2 Unverified Assumptions

There are no unverified assumptions.

5.0 Design Inputs

5.1 Original Compressive Strength Data

The original compressive strength data were used to determine the original elastic modulus using Equation 1. Seabrook Station provided MPR with Concrete Compressive Strength Test Reports from Pittsburgh Testing Laboratory (Reference 5 and Reference 9). These lab reports contained the 28-day compressive strength data from cylinders that were representative of the majority of the locations of interest. The cylinders used to determine the 28-day compressive strength were molded using concrete from the same concrete batch that was used to place the associated concrete structure at Seabrook Station.

Average compressive strength values for specific structures provided in Reference 4 were used when applicable Concrete Compressive Strength Test Reports from Pittsburgh Testing Laboratory were not available. Reference 4 evaluates available 28-day compressive strength values of concrete cylinders during the original construction of Seabrook Station.
The calculation determines the average of all compressive strength values and calculates the range and standard deviation. Using the average compressive strength value for Seabrook Station (Reference 4) for locations that do not have applicable test reports is appropriate due to the fact that original compressive strength does not have a significant effect on the through-thickness expansion to-date.

Table 5-1 presents the average and standard deviation associated with the original compressive strength of each location. The average compressive strength is used to determine the nominal through-thickness expansion to-date. The range and standard deviation illustrate the variability among the original compressive strength data.

Table 5-1. Original Compressive Strength Data

Location ID Note 1	Average (psi)	Range (psi)	Standard Deviation (psi)	Reference
E1	5197	1240	371	5
E2	6163	2140	705	5
E3	5666	1000	320	5
E4	4429	1750	526	5
E5	5266	880	363	5
E6	5922	1200	401	5

Table 5-1. Original Compressive Strength Data

Location ID Note 1	Average (psi)	Range (psi)	Standard Deviation (psi)	Reference
E7	6412	780	217	5
E8	5426	980	315	5
E9	4910	1510	400	5
E10	5186	870	243	5
E11	5774	1700	530	5
E12	5666	1000	320	5
E13	5710	180	104	5
E14	5426	980	315	5
E15	6037	170	93	5
E18	5456	3120	568	4
E19	5456	3120	568	4
E20	5307	1820	528	9
E21	5490	1260	381	9
E22	5456	3120	568	4
E23	5660	710	254	9
E24	5456	3120	568	4
E25	5537	1100	332	9
E26	5390	920	257	9
E28	5260	2720	821	9
E29	5662	1860	558	9
E30	5662	1860	558	9
E31	5456	3120	568	4

Table 5-1. Original Compressive Strength Data

Location ID Note 1	Average (psi)	Range (psi)	Standard Deviation (psi)	Reference
E32	5133	2060	461	9
E33	5106	3340	822	9
E35	4997	300	101	9
E36	5456	3120	568	4
E37	5456	3120	568	4
E39	5426	980	306	9
E40	5346	1980	470	9
E41	5456	3120	568	4
E42	5348	640	218	9
E43	5348	640	218	9

Notes:
1. Locations E16, E17, E27, E34, and E38 were deleted from the original scope. Thus, extensometers were not installed at these locations.

5.2 *Current Elastic Modulus Data*

Seabrook Station determined the current elastic modulus by testing cores removed from each location and provided the results to MPR (Reference 6, 7, and 8). Results from these tests are listed in Table 5-2.

In the majority of locations, multiple elastic modulus values were obtained. The "-1," "-2," "-3," and "-4" after the location title designate between the specific core locations. Some locations have multiple modulus results because sufficient intact core length was available for two test specimens. The average and range values presented below consider all tests performed on cores from the same general location. The average current elastic modulus data is used to determine the nominal through-thickness expansion to-date. The range illustrates the variability associated with current modulus data.

In some locations, field conditions (e.g., cracked cores) and configuration limitations (e.g., embedded steel and conduits, rebar, etc.) limited the number of cores that could be obtained and tested. In these cases, only one elastic modulus value was obtained. These locations are identified with a range of "N/A" in Table 5-2.

It is preferred that Seabrook Station obtain at least two elastic modulus test results from each location of interest and average the results to promote greater accuracy. MPR reviewed the reported elastic modulus values and noted the following:

- All single elastic modulus values are within the range of average elastic modulus values from other locations. This observation suggests that the concrete in locations with only one modulus value is in comparable condition to other locations within the plant, which provides assurance that the values are reasonable.

- Of the eleven locations with only one elastic modulus value, nine have calculated nominal expansion values that are very low (i.e., 0.07%, See Table 6-2). Therefore, the effects of minor inaccuracies associated with the elastic modulus obtained at these locations are insignificant. NextEra is further investigating the two locations with higher nominal through-thickness values.

Table 5-2. Current Elastic Modulus Data

Location ID Note 1	Modulus 1 (psi)	Modulus 2 (psi)	Average (psi)	Range (psi)
E1-1	2.20E+06	2.10E+06	2.04E+06	8.50E+05
E1-2	2.35E+06	1.50E+06		
E2-1	3.00E+06	N/A	2.70E+06	6.00E+05
E2-2	2.40E+06	N/A		
E3-1	2.35E+06	2.10E+06	2.49E+06	7.00E+05
E3-2	2.80E+06	2.70E+06		
E4-1	2.80E+06	N/A	3.30E+06	1.00E+06
E4-2	3.80E+06	N/A		
E5-1	4.45E+06	N/A	4.53E+06	1.50E+05

Table 5-2. Current Elastic Modulus Data

Location ID Note 1	Modulus 1 (psi)	Modulus 2 (psi)	Average (psi)	Range (psi)
E5-2	4.60E+06	N/A		
E6-1	2.95E+06	2.90E+06	2.91E+06	2.00E+05
E6-2	3.00E+06	2.80E+06		
E7-1	3.15E+06	3.05E+06	2.97E+06	4.50E+05
E7-2	2.70E+06	N/A		
E8-1	2.40E+06	N/A	2.55E+06	3.00E+05
E8-2	2.70E+06	N/A		
E9-1	1.40E+06	1.80E+06	1.50E+06	5.00E+05
E9-2	1.30E+06	N/A		
E10-1	2.20E+06	2.30E+06	2.41E+06	4.00E+05
E10-2	2.50E+06	2.45E+06		
E10-2 (cont.)	2.60E+06	N/A		
E11-1	2.75E+06	N/A	2.83E+06	1.5E+05
E11-3	2.90E+06	N/A		
E12-1	3.10E+06	3.05E+06	3.16E+06	4.00E+05
E12-2	3.45E+06	3.05E+06		
E13-1	N/A	N/A	1.85E+06	N/A
E13-2	1.85E+06	N/A		

Table 5-2. Current Elastic Modulus Data

Location ID Note 1	Modulus 1 (psi)	Modulus 2 (psi)	Average (psi)	Range (psi)
E14-1	2.25E+06	1.90E+06	1.88E+06	6.00E+05
E14-2	1.70E+06	1.65E+06		
E15-1	2.25E+06	N/A	2.38E+06	2.50E+05
E15-2	2.50E+06	N/A		
E18-1	2.85E+06	N/A	2.98E+06	2.50E+05
E18-2	3.10E+06	N/A		
E19-1	3.10E+06	N/A	3.38E+06	5.50E+05
E19-2	3.65E+06	N/A		
E20-1	3.50E+06	N/A	3.55E+06	1.00E+05
E20-2	3.60E+06	N/A		
E21-1	1.05E+06	1.40E+06	1.50E+06	8.50E+05
E21-2	1.65E+06	1.90E+06		
E22-1	3.95E+06	N/A	3.95E+06	N/A
E22-2	N/A	N/A		
E23-3	N/A	N/A	3.05E+06	N/A
E23-4	3.05E+06	N/A		
E24-1	N/A	N/A	2.95E+06	N/A
E24-2	2.95E+06	N/A		

Table 5-2. Current Elastic Modulus Data

Location ID Note 1	Modulus 1 (psi)	Modulus 2 (psi)	Average (psi)	Range (psi)
E25-1	4.75E+06	N/A	4.98E+06	4.50E+05
E25-2	5.20E+06	N/A		
E26-1	2.25E+06	2.70E+06	2.58E+06	8.00E+05
E26-2	2.30E+06	3.05E+06		
E28-1	4.10E+06	N/A	4.10E+06	N/A
E28-2	N/A	N/A		
E29-1	3.75E+06	N/A	3.68E+06	1.50E+05
E29-2	3.60E+06	N/A		
E30-1	1.90E+06	2.80E+06	2.58E+06	1.15E+06
E30-2	2.55E+06	3.05E+06		
E31-1	2.30E+06	2.40E+06	3.32E+06	3.10E+06
E31-2	5.40E+06	3.90E+06		
	2.60E+06	N/A		
E32-1	2.20E+06	N/A	2.35E+06	3.50E+05
E32-2	2.55E+06	2.30E+06		
E33-1	3.05E+06	N/A	3.05E+06	0.00E+00
E33-2	3.05E+06	N/A		
E35-1	2.50E+06	N/A	2.50E+06	N/A
E35-2	N/A	N/A		

Table 5-2. Current Elastic Modulus Data

Location ID Note 1	Modulus 1 (psi)	Modulus 2 (psi)	Average (psi)	Range (psi)
E36-1	N/A	N/A	4.60E+06	N/A
E36-2	4.60E+06	N/A		
E37-1	N/A	N/A	3.05E+06	N/A
E37-2	3.05E+06	N/A		
E39-1	2.25E+06	N/A	2.75E+06	1.00E+06
E39-2	3.25E+06	N/A		
E40-1	N/A	N/A	2.78E+06	3.50E+05
E40-2	2.95E+06	2.60E+06		
E41-3	4.00E+06	N/A	4.00E+06	N/A
E41-4	N/A	N/A		
E42-1	1.60E+06	N/A	1.60E+06	N/A
E42-2	N/A	N/A		
E43-1	N/A	N/A	2.75E+06	N/A
E43-2	N/A	N/A		
E43-3	2.75E+06	N/A		

Notes:
1. Locations E16, E17, E27, E34, and E38 were deleted from the original scope. Thus, extensometers were not installed at these locations.

6.0 Calculations and Results

6.1 Original Elastic Modulus

The original elastic modulus was determined by using the average compressive strength data in Table 5-1 and Equation 1, where f_c' is the 28-day compressive strength and E_c is the original elastic modulus. Results are presented in Table 6-1.

Table 6-1. Nominal Original Elastic Modulus

Location ID Note 1	Original Elastic Modulus (psi)
E1	4.11E+06
E2	4.47E+06
E3	4.29E+06
E4	3.79E+06
E5	4.14E+06
E6	4.39E+06
E7	4.56E+06
E8	4.20E+06
E9	3.99E+06
E10	4.10E+06
E11	4.33E+06
E12	4.29E+06
E13	4.31E+06
E14	4.20E+06

Table 6-1. Nominal Original Elastic Modulus

Location ID Note 1	Original Elastic Modulus (psi)
E15	4.43E+06
E18	4.21E+06
E19	4.21E+06
E20	4.15E+06
E21	4.22E+06
E22	4.21E+06
E23	4.29E+06
E24	4.21E+06
E25	4.24E+06
E26	4.18E+06
E28	4.13E+06
E29	4.29E+06
E30	4.29E+06
E31	4.21E+06
E32	4.08E+06
E33	4.07E+06
E35	4.03E+06
E36	4.21E+06
E37	4.21E+06

Table 6-1. Nominal Original Elastic Modulus

Location ID Note 1	Original Elastic Modulus (psi)
E39	4.20E+06
E40	4.17E+06
E41	4.21E+06
E42	4.17E+06
E43	4.17E+06

Notes:

1. Locations E16, E17, E27, E34, and E38 were deleted from the original scope. Thus, extensometers were not installed at these locations.

6.2 Nominal Through-Thickness Expansion To-Date

The average modulus values presented in Table 5-2 and the nominal original elastic modulus values listed in Table 6-1 were used to determine the normalized modulus (*modulus*).
The nominal expansion to-date was calculated using the normalized modulus and Equation 3.

[redacted] (Equation 3)

The nominal through-thickness expansion values (i.e., unadjusted though-thickness expansion values) to-date for the locations of interest are presented in Table 6-2.

Table 6-2. Nominal Through-Thickness Expansion To-Date

[redacted]

MPR

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320 King
Street
Alexandria, VA 22314

Prepared By: Amanda Card

Checked By: [signature]

Calculation No.:
0326-0062-CLC-04

Revision No.: 1

Table 6-2. Nominal Through-Thickness
Expansion To-Date

[redacted]

MPR

MPR Associates, Inc.
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Alexandria, VA 22314

Prepared By: Amanda Card

Checked By: [signature]

Calculation No.:
0326-0062-CLC-04

Revision No.: 1

Table 6-2. Nominal Through-Thickness
Expansion To-Date

[redacted]

6.3 *Adjusted Through-Thickness Expansion To-Date*

Uncertainty in the original modulus (calculated from the original compressive strength) and the measurement variability in current modulus influence the calculated through-thickness expansion values.

To include an appropriate level of conservatism into the calculated through-thickness values, a normalized modulus reduction factor of 0.85 was applied, as shown in Equation 4 below.

[redacted] (Equation 4)

Equation 4 results in higher calculated through-thickness values. Results for the locations of interest are shown in Table 6-3. The average original compressive strength, the calculated original elastic modulus, the average current elastic modulus, and the nominal through-thickness expansion values are included for reference.

Table 6-3. Through-Thickness Expansion To-Date

Location ID	Average Original Compressive Strength (psi)	Original Elastic Modulus (psi)	Average Current Elastic Modulus (psi)	Nominal Through-Thickness Expansion	Through-Thickness Expansion ([redacted] factor)
E1	5197	4.11E+06	2.04E+06	[redacted]	[redacted]
E2	6163	4.47E+06	2.70E+06	[redacted]	[redacted]
E3	5666	4.29E+06	2.49E+06	[redacted]	[redacted]
E4	4429	3.79E+06	3.30E+06	[redacted]	[redacted]
E5	5266	4.14E+06	4.53E+06	[redacted]	[redacted]
E6	5922	4.39E+06	2.91E+06	[redacted]	[redacted]
E7	6412	4.56E+06	2.97E+06	[redacted]	[redacted]
E8	5426	4.20E+06	2.55E+06	[redacted]	[redacted]
E9	4910	3.99E+06	1.50E+06	[redacted]	[redacted]
E10	5186	4.10E+06	2.41E+06	[redacted]	[redacted]
E11	5774	4.33E+06	2.83E+06	[redacted]	[redacted]

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Table 6-3. Through-Thickness Expansion To-Date

Location ID	Average Original Compressive Strength (psi)	Original Elastic Modulus (psi)	Average Current Elastic Modulus (psi)	Nominal Through-Thickness Expansion	Through-Thickness Expansion ([redacted] factor)
E12	5666	4.29E+06	3.16E+06	[redacted]	[redacted]
E13	5710	4.31E+06	1.85E+06	[redacted]	[redacted]
E14	5426	4.20E+06	1.88E+06	[redacted]	[redacted]
E15	6037	4.43E+06	2.38E+06	[redacted]	[redacted]
E18	5456	4.21E+06	2.98E+06	[redacted]	[redacted]
E19	5456	4.21E+06	3.38E+06	[redacted]	[redacted]
E20	5307	4.15E+06	3.55E+06	[redacted]	[redacted]
E21	5490	4.22E+06	1.50E+06	[redacted]	[redacted]
E22	5456	4.21E+06	3.95E+06	[redacted]	[redacted]
E23	5660	4.29E+06	3.05E+06	[redacted]	[redacted]
E24	5456	4.21E+06	2.95E+06	[redacted]	[redacted]
E25	5537	4.24E+06	4.98E+06	[redacted]	[redacted]
E26	5390	4.18E+06	2.58E+06	[redacted]	[redacted]
E28	5260	4.13E+06	4.10E+06	[redacted]	[redacted]
E29	5662	4.29E+06	3.68E+06	[redacted]	[redacted]
E30	5662	4.29E+06	2.58E+06	[redacted]	[redacted]
E31	5456	4.21E+06	3.32E+06	[redacted]	[redacted]
E32	5133	4.08E+06	2.35E+06	[redacted]	[redacted]
E33	5106	4.07E+06	3.05E+06	[redacted]	[redacted]
E35	4997	4.03E+06	2.50E+06	[redacted]	[redacted]
E36	5456	4.21E+06	4.60E+06	[redacted]	[redacted]

Table 6-3. Through-Thickness Expansion To-Date

Location ID	Average Original Compressive Strength (psi)	Original Elastic Modulus (psi)	Average Current Elastic Modulus (psi)	Nominal Through-Thickness Expansion	Through-Thickness Expansion ([redacted] factor)
E37	5456	4.21E+06	3.05E+06	[redacted]	[redacted]
E39	5426	4.20E+06	2.75E+06	[redacted]	[redacted]
E40	5346	4.17E+06	2.78E+06	[redacted]	[redacted]
E41	5456	4.21E+06	4.00E+06	[redacted]	[redacted]
E42	5348	4.17E+06	1.60E+06	[redacted]	[redacted]
E43	5348	4.17E+06	2.75E+06	[redacted]	[redacted]

The results in Table 6-3 indicate that Equation 4 inherently provides significant conservatism. Key observations include the following:

- For the highest through-thickness expansion value of [redacted]% (location E21), use of Equation 4 increased the expansion value to [redacted]% (i.e., [redacted]% expansion). The impact of the normalized modulus reduction factor (in absolute terms) increases with ASR progression (i.e., at higher levels of expansion).

- In relative terms, application of Equation 4 to the highest through-thickness expansion value (location E21) produced a conservatism of [redacted]% (i.e., [redacted]% expansion / [redacted]% expansion).

- The relative conservatism of Equation 4 increases if ASR progression is less advanced. As an example, for location E1, where nominal expansion is [redacted]%, the relative conservatism of using Equation 4 is [redacted]% (i.e., [redacted]% expansion/ [redacted]% expansion).

7.0 REFERENCES

1. MPR Calculation 0326-0062-CLC-01, *Evaluation of ACI Equation for Elastic Modulus*, Revision 0.

2. ACI 318-71, "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, 1971.

3. MPR Calculation 0326-0062-CLC-03, *Correlation Between Through-Thickness Expansion and Elastic Modulus in Concrete Test Specimens Affected by Alkali-Silica Reaction (ASR)*, Revision 3.

4. MPR Calculation 0326-0062-CLC-02, *Compressive Strength Values for Concrete at Seabrook Station*, Revision 0.

5. Pittsburg Testing Laboratory Concrete Compressive Strength Test Reports, transmitted to MPR from Seabrook Station via SBK-L-16086, "Documentation Transmittal to Support Determination of Through-Thickness Expansion to Date and Validate Expansion Behavior at Seabrook," June 9, 2016.

6. Simpson Gumpertz & Heger Report No. 160072-LR-01, Revision 0, "Laboratory Testing of Concrete Cores at SGH, NextEra Energy Seabrook Station, Waltham, MA," May 3, 2016.

7. Simpson Gumpertz & Heger Document No. 160072.02-L-001, "Onsite Support and Testing of Twenty-Eight Cores, NextEra Energy Seabrook Station, Waltham, MA," July 15, 2016.

8. Correlation Cores Status Matrix, transmitted to MPR from Seabrook Station via SBK-L-17110, "Documentation Transmittal to Support Determination of Through-Thickness Expansion to Date and Validate Expansion Behavior at Seabrook," July 28, 2017.

9. Pittsburg Testing Laboratory Concrete Compressive Strength Test Reports, transmitted to MPR from Seabrook Station via SBK-L-17110, "Documentation Transmittal to Support Determination of Through-Thickness Expansion to Date and Validate Expansion Behavior at Seabrook," July 28, 2017.

Enclosure 5 to SBK-L-18072

MPR-4273, Revision 1, "Seabrook Station – Implication of Large-Scale Test Program Results on Reinforced Concrete Affected by ASR," March 2018 (Seabrook FP# 101050);

(Non-proprietary)

-- Non-Proprietary Version --

MPR

MPR-4273
Revision 1
(Seabrook FP# 101050)
March 2018

Seabrook Station - Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction

QUALITY ASSURANCE DOCUMENT
This document has been prepared, reviewed, and approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B and/or ASME NQA-1, as specified in the MPR Nuclear Quality Assurance Program.

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-- Non-Proprietary Version --

MPR

Seabrook Station - Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction

MPR-4273
(Seabrook FP# 101050)
Revision 1
March 2018

QUALITY ASSURANCE DOCUMENT
This document has been prepared, reviewed, and approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B and/or ASME NQA-1, as specified in the MPR Nuclear Quality Assurance Program.

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RECORD OF REVISIONS

Revision	Affected Pages	Description
0	All	Initial Issue
1	Executive Summary, Section 6, and Appendices B and C	Updated to reflect refinements to MPR's original recommendations for assessing expansion behavior at Seabrook Station, and to incorporate additional guidance to support implementation of recommended approach at Seabrook Station. Made minor editorial edits throughout.

Acknowledgements

This report documents large-scale test programs conducted to support evaluation of the impact of alkali-silica reaction on reinforced concrete structures at Seabrook Station. The test programs were a collaborative effort between MPR Associates and the Ferguson Structural Engineering Laboratory (FSEL) (which is part of The University of Texas at Austin). These programs required a large team of engineers and researchers, and countless man-hours over a four-year period. Successful completion of such an ambitious project is a testament to the dedication, commitment, and technical contributions of the entire MPR/FSEL team, and active engagement and support by NextEra Energy.

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Executive Summary

On behalf of NextEra, MPR directed several large-scale test programs to investigate the structural impact of alkali-silica reaction (ASR) on reinforced concrete specimens. The test programs involved fabrication and testing of [redacted] large-scale test specimens that were designed to represent reinforced concrete structures at Seabrook Station and testing of two ASR-affected bridge girders. Testing included [redacted] anchor capacity tests, [redacted] shear load tests, [redacted] flexural load tests, and evaluation of [redacted] instrument configurations (total of [redacted] instruments) for monitoring through-thickness expansion. This report integrates the conclusions of those studies to present the implications for structural assessments and monitoring of reinforced concrete structures at the plant, as follows:

- ASR causes expansion of affected concrete that initially proceeds in all directions regardless of reinforcement configuration. The two-dimensional reinforcement mats in the test specimens confined expansion in the plane of the reinforcement mats (i.e., the in-plane directions) after [redacted] expansion. Subsequent expansion was primarily in the through-thickness direction. The reinforcement configuration of the test specimens reflects Seabrook Station structures. Accordingly, in-plane expansion measurements at Seabrook are sufficient for monitoring ASR progression until expansion reaches [redacted], after which through-thickness expansion measurements are necessary.

- The Combined Cracking Index (CCI) methodology (and the Seabrook Station procedure, in particular) provides a reasonable approximation of true engineering strain and is an acceptable methodology for in-plane expansion monitoring.

- Snap ring borehole extensometers (SRBEs) provide an accurate and reliable methodology for monitoring through-thickness expansion from the time the SRBE is installed.

- To determine total through-thickness expansion, NextEra will also need to identify the through-thickness expansion before the SRBE is installed. The test programs identified that elastic modulus is sensitive to ASR degradation and provides a repeatable correlation with through-thickness expansion. Through-thickness expansion determined from the empirical correlation may be added to the SRBE-determined expansion to calculate the total through-thickness expansion. (See MPR-4153 for details.)

- Results from the Anchor Test Program indicate that there is no reduction in anchor capacity in ASR-affected concrete with in-plane expansion levels of less than [redacted] mm/m [redacted]. Because in-plane expansion of fabricated test specimens plateaued at [redacted] expansion, anchor testing was performed on two ASR-affected bridge girders to investigate anchor performance at higher expansion levels. Anchor capacity is insensitive to through-thickness expansion and time of installation relative to ASR expansion (i.e., installed before or after the onset of expansion).

- Results from the Shear Test Program indicate that there is no reduction of shear capacity in ASR-affected concrete with through-thickness expansion levels up to [redacted], which was the maximum ASR expansion level exhibited by shear test specimens. (Test results show that the shear capacity actually increases due to pre-stressing from ASR expansion, but MPR recommends that this "benefit" should not be credited.)

- Results from the Reinforcement Anchorage Test Program indicate that there is no reduction in the performance of reinforcement lap splices in ASR-affected concrete with through-thickness expansion levels up to [redacted], which was the maximum ASR expansion level exhibited by reinforcement anchorage test specimens.

- The progression of ASR in the reinforcement anchorage test specimens resulted in a notable change in stiffness, characterized by a decrease in deflection at yield. The increase in stiffness is due to pre-stressing from ASR expansion.

A companion report (MPR-4288, "Seabrook Station: Impact of Alkali-Silica Reaction on the Structural Design Basis") describes the effect of ASR on the structural design basis of affected structures at Seabrook Station and provides guidance for evaluations of those structures. Content from this report provides evaluation criteria for selected limit states (shear, reinforcement anchorage, anchor capacity).

Execution of a multi-year large-scale test program to support evaluation of ASR-affected reinforced concrete structures is unique in the nuclear industry in purpose, scale, and methodology. Application of the results of the FSEL test programs requires that the test specimens be representative of reinforced concrete at Seabrook Station, and that expansion behavior of concrete at the plant be similar to that observed in the test specimens. Test specimen design addressed representativeness of the test specimens, and promoted expansion behavior consistent with the plant (e.g., use of two-dimensional reinforcement mats). To confirm that expansion behavior at Seabrook Station is similar to the FSEL test specimens, this report recommends that NextEra perform the checks identified in the table below. Appendices B and C provide detailed procedures to support implementation of the recommended approach at Seabrook Station.

Table 1. Recommendations for Confirming Expansion Behavior at Seabrook Station is Similar to Test Programs

Objective	Recommended Approach	When
Ongoing Monitoring		
Expansion within limits from test programs	Compare measured in-plane expansion (ε_{xy}), through-thickness expansion (ε_z), and volumetric expansion (ε_v) at the plant to limits from test programs ($\varepsilon_{xy} \leq$ [redacted]%, $\varepsilon_z \leq$ [redacted]%, and $\varepsilon_v <$ [redacted]%)	Intervals as specified in Structures Monitoring Program (SMP) or Aging Management Program (AMP)
Lack of mid-plane crack	Inspect cores removed from ASR-affected structures (and boreholes) for evidence of mid-plane cracks	When cores are removed to install extensometers or for other reasons.
Periodic Confirmation of Expansion Behavior		
Lack of mid-plane crack	Review of records for cores removed to date or since last assessment	Periodic assessments • At least 5 years prior to the Period of Extended Operations (PEO) • Every 10 years thereafter
Expansion initially similar in all directions but becomes preferential in z-direction	Compare ε_{xy} to ε_z using a plot of ε_z versus in-plane expansion	
Expansions within range observed in test programs	Compare measured ε_{xy}, ε_z, and ε_v at the plant to limits from test programs ($\varepsilon_{xy} \leq$ [redacted]%, $\varepsilon_z \leq$ [redacted]%, and $\varepsilon_v <$ [redacted]%) to check margin for future expansion	
Corroborate modulus-expansion correlation with plant data	For 20% of the extensometer locations: • Remove cores for modulus testing. • Compare ε_z determined from the modulus-expansion correlation with ε_z determined from the extensometer and the original modulus result. A detailed explanation of this approach is provided in Appendix C.	At least 5 years prior to PEO (initial study) and 10 years thereafter (follow-up study).

Contents

Contents (cont'd.)

Tables

Figures

AEC 5/18/2018
JAS 5-18-2018
CWB 5-18-2018
4v 5-18-2018

Figures (cont'd.)

EC
/18/2018
JWS
5-18-2018
CWB
-18-2018
-18-2018

Figures (cont'd.)

EC
5/18/2018
JvS
5-18-2018
CWB
5-18-2018
5-18-2018

1 Introduction

1.1 Purpose

On behalf of NextEra, MPR directed several large-scale test programs to investigate the structural impact of Alkali Silica Reaction (ASR) on reinforced concrete specimens. This report integrates the conclusions of those studies to present the implications for structural assessments and monitoring of reinforced concrete structures at the plant.

1.2 Background

1.2.1 Alkali-Silica Reaction

ASR occurs in concrete when reactive silica 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, exerting tensile stress on the surrounding concrete and resulting in cracking. Typical cracking caused by ASR is described as "pattern" or "map" cracking and is usually accompanied by dark staining adjacent to the cracks. Figure 1-1 provides an illustration of this process.

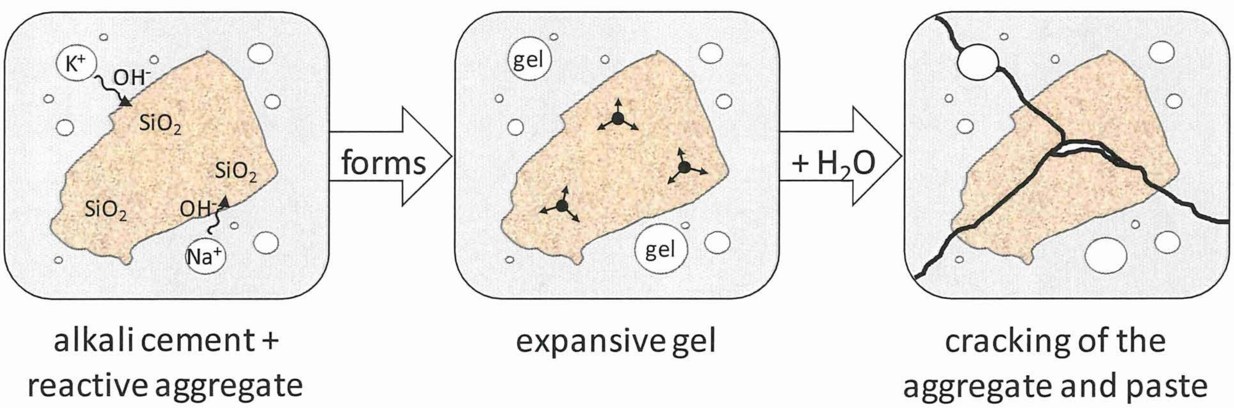

Figure 1-1. ASR Expansion Mechanism

The cracking may degrade the material properties of the concrete, necessitating an assessment of the adequacy of the affected structures and supports anchored to the structures.

1.2.2 ASR at Seabrook Station

NextEra has identified ASR in multiple safety-related, reinforced concrete structures at Seabrook Station (Reference 1.1). After an extent of condition determination that identified potentially affected structures at the site, MPR performed an interim structural assessment (Reference 2.1) of selected ASR-affected structures to evaluate their adequacy given the presence of ASR. Based on the low level of observed cracking and the apparent slow rate of change, MPR concluded that these structures are suitable for continued service for at least an interim period (i.e., at least several years).

The interim structural assessment (Reference 2.1) utilized a conservative treatment of data from existing literature, supplemented by limited testing of anchor bolts, to produce conclusions suitable for a short-term structural assessment. NextEra will perform follow-up evaluations to assess the long-term adequacy of the concrete structures and attachments at Seabrook Station. In support of these evaluations, MPR conducted large-scale test programs of specimens that were designed and fabricated to represent reinforced concrete at Seabrook Station to the maximum extent practical. Results from the large-scale test programs provide input to determine the potential effects of ASR on adequacy of structures at Seabrook Station.

Because the design codes for Seabrook Station do not include provisions for ASR, NextEra is submitting a License Amendment Request (LAR) to incorporate a methodology for evaluating ASR-affected structures into the plant's licensing basis. This report provides the technical basis for portions of the LAR that were developed from the results of the large-scale test programs.

Figure 1-2 provides a high-level summary of the key activities of the ASR project at Seabrook Station related to evaluation of structural capacity of ASR-affected structures[1].

Figure 1-2. Activities for Evaluating Structural Capacity of ASR-Affected Structures

1.2.3 Test Programs at FSEL

MPR directed four test programs at the Ferguson Structural Engineering Laboratory (FSEL) at The University of Texas at Austin (UT-Austin) to support NextEra's efforts to resolve the ASR issue identified at Seabrook Station. Three of the test programs focused on the structural performance data necessary to complete the follow-up structural evaluations of ASR-affected structures. The fourth test program evaluated instruments for monitoring expansion at Seabrook Station.

In each structural test program, ASR developed in the fabricated test specimens and was routinely monitored so that testing could be performed at particular levels of ASR distress. This approach enabled systematic development of trends for structural performance with the

[1] The LAR will include the methodology for the final structural assessment; the actual assessment may be completed after submittal of the LAR.

progression of ASR. The resulting data sets were a significant improvement upon the collection of published literature sources, because test data across the range of ASR distress levels were obtained using a common methodology and identical test specimens.

A brief overview of each test program is provided below.

- Anchor Test Program – This test program evaluated the impact of ASR on performance of expansion anchors and undercut anchors installed in concrete. Test specimens included [redacted] large-scale blocks that were designed and fabricated to represent the reinforced concrete structures at Seabrook Station and two sections of a reinforced concrete bridge girder that was available at FSEL. The test program consisted of a total of [redacted] anchor tests. (Reference 4.1)

- Shear Test Program – This test program evaluated the impact of ASR on shear capacity of reinforced concrete specimens. Three-point load tests were performed on large-scale beams that were designed and fabricated to represent the reinforced concrete structures at Seabrook Station. FSEL fabricated [redacted] shear test specimens and conducted a total of [redacted] tests (two tests performed on most specimens). (Reference 4.2)

- Reinforcement Anchorage Test Program – This program evaluated the impact of ASR on reinforcement anchorage of rebar lap splices embedded in concrete and also provided insights on flexural strength and stiffness. Four-point load tests were performed on large-scale beams that were designed and fabricated to represent the reinforced concrete structures at Seabrook Station. FSEL fabricated [redacted] reinforcement anchorage test specimens and conducted a total of [redacted] tests (one test per specimen). (Reference 4.2)

- Instrumentation Test Program – This program evaluated instruments for the measurement of through-thickness expansion. Insights gained from this program were used to select which instrument to use at Seabrook Station and to refine installation procedures. The test specimen was a large-scale reinforced concrete beam that was designed and fabricated to represent reinforced concrete structures at Seabrook Station. Testing included a total of [redacted] instruments over [redacted] different configurations. FSEL periodically monitored expansion using these instruments for one year. (Reference 4.3)

1.2.4 Additional Testing

The Anchor, Shear, Reinforcement Anchorage, and Instrumentation Test Programs were designed to produce data that would ultimately be used as inputs for safety-related evaluations at Seabrook Station. Additional testing was performed to inform decisions on directing these test programs and provide insights that help interpret test program results.

Expansion Behavior

As part of each test program, expansion of the test specimens was monitored in a variety of ways to characterize ASR progression. An additional study was performed outside the scope of the test programs that focused on monitoring the total axial and volumetric expansion of concrete cubes with varying reinforcement layouts, reinforcement density, and concrete mix designs.

This additional study provides insights on the factors for expansion behavior and their relative importance. (Reference 6.1)

Retrofit Testing

For the Shear and Reinforcement Anchorage Test Programs, the original intent was to develop ASR and perform tests until a threshold for ASR distress was identified where structural performance declined. FSEL would then install retrofits to specimens at higher ASR levels (e.g., by installing grouted rods to function like shear reinforcement) and perform load testing to qualify a repair methodology. Proof-of-concept testing of candidate retrofits was performed using specimens that were not affected by ASR[2]. (References 6.2 & 6.3)

Uniform Load Testing

The load test setup for the Shear Test Program used a hydraulic ram and two beam supports to apply three-point loading. Use of point loads is convenient, but a uniform distribution would be more representative of the loads applied to some actual structures (e.g., hydrostatic loading on the exterior surface of a below-grade wall). FSEL performed uniform load shear testing on specimens with a design comparable to the specimens for the Shear Test Program to assess the difference in shear capacity for the different loading conditions. The load test setup for the uniform load tests applied force using an air bladder to exert uniform pressure to the underside of each specimen. (References 6.4 & 6.5)

1.3 Commercial Grade Dedication

The test programs were performed by FSEL with technical direction and quality assurance oversight from MPR. The testing was governed by MPR test specifications (References 3.1 & 3.2) and was 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 (CGD) Reports for the Anchor, Shear, Reinforcement Anchorage, and Instrumentation Test Programs (References 5.1, 5.2, 5.3, & 5.4).

The additional studies on expansion behavior of concrete cubes, retrofit testing on non-ASR affected specimens, and uniform load distribution were not commercially dedicated. Conclusions from these efforts inform the overall project, but were not used to develop quantitative inputs for evaluation of structures at Seabrook Station.

1.4 Report Scope

This report combines the key conclusions from the four test programs, results from the additional testing studies, and information gathered as part of MPR's overall investigation of ASR at Seabrook Station to provide integrated conclusions that support NextEra's follow-up structural evaluations and monitoring of ASR-affected reinforced concrete. Detailed information on the specimen designs, test methods, and test results are provided in the test program reports (References 4.1, 4.2, & 4.3), which provide complete documentation of the test programs.

[2] Ultimately, the retrofits were not tested on ASR-affected specimens, because structural testing of ASR-affected specimens without retrofits did not identify a decrease in structural performance for the ASR levels that were achievable within the duration of the test programs.

Further information on the additional testing studies is provided in UT-Austin documents (References 6.1, 6.2, 6.3, 6.4, & 6.5).

Table 1-1 summarizes the primary source documentation for test results from the MPR/FSEL test programs.

Table 1-1. Summary of Support Documentation

Test Program	Test Reports	CGD Reports
Anchor	MPR-3722 (Reference 4.1)	MPR-3726 MPR-4247 MPR-4286 (References 5.1, 5.2, & 5.4)
Shear	MPR-4262 (Reference 4.2)	MPR-4259 MPR-4286 (References 5.3 & 5.4)
Reinforcement Anchorage		
Instrumentation	MPR-4231 (Reference 4.3)	
Information Only	UT-Austin Documentation (References 6.1, 6.2, 6.3, 6.4, & 6.5)	N/A

A companion report (MPR-4288, "Seabrook Station: Impact of Alkali-Silica Reaction on the Structural Design Basis") describes the effect of ASR on the structural design basis of affected structures at Seabrook Station and provides guidance for evaluations of those structures.

2 Selection of Approach for Test Programs

This section highlights the reasons for pursuing the MPR/FSEL test programs and summarizes the rationale for key decisions that shaped and focused the approach for testing. The key decision points were as follows:

- Focus on structural testing to capture the interplay between ASR expansion and the restraint provided by the reinforcement (i.e., confinement).
- Address limit states of interest for structures at Seabrook Station where there were limitations or gaps in the available literature, especially where available margins are low or the apparent effect of ASR is high.
- Use laboratory-prepared test specimens to facilitate separate effects studies to determine the impact of ASR on structural performance as a function of the severity of ASR.
- Ensure results are applicable to structures at Seabrook Station by designing specimens to be representative and using test approaches consistent with those used to calibrate the code equations.

The decisions that defined the test program were informed by a comprehensive review of literature on ASR degradation and its impacts on structural performance. The literature review and the key decision points are discussed below.

2.1 Summary of Literature Review

As part of developing the approach for addressing ASR-affected concrete at Seabrook Station, MPR conducted a comprehensive review of published research on the structural implications of ASR and industry guidance for evaluating ASR-affected structures. Most research on ASR has focused on the science and kinetics of ASR, rather than engineering research on structural implications. Structural testing of ASR-affected test specimens has been performed, but application of the conclusions to a specific structure can be challenged by lack of representativeness.

Industry guidelines from the Institution of Structural Engineers (Reference 1.2) and the Federal Highway Administration (Reference 1.3) provide a summary of potential implications of ASR and high level information that MPR used to identify focus areas for addressing ASR at Seabrook Station. MPR's literature review included over a hundred detailed references to explore approaches for evaluating ASR-affected structures. These efforts led to the initial series of actions at Seabrook Station including petrographic examinations to confirm the presence of ASR, extent of condition walkdowns that utilized crack width summation to quantitatively characterize the effect of ASR, and development of a protocol for monitoring further development of ASR during the ongoing project.

The literature also established the expectation for a reduction in material properties of cores from ASR-affected concrete, and identified that such a reduction does not necessarily reflect a corresponding decrease in structural capacity. The presence of two-dimensional reinforcement mats at Seabrook Station provides confinement that differentiates structural performance from un-reinforced concrete structures (e.g., dams) that are more appropriately represented by cores. ASR-induced expansion in reinforced concrete has a "prestressing" effect that mitigates loss of structural capacity.

A focused review of published research on the structural implications of ASR (Reference 2.2) identified dozens of technical references on testing of ASR-affected concrete. The most relevant references were used to support the interim structural assessment for Seabrook Station by providing a conservatively bounding capacity reduction factor for structural limit states (e.g., shear) to account for the presence of ASR. For these technical papers, Reference 2.2 discussed the extent to which the experimental design and test specimens were representative of structures with two-dimensional reinforcement (like structures at Seabrook Station). For completeness, Reference 2.2 also identified testing of ASR-affected concrete that was poorly representative of Seabrook Station and why it should not be used for a structural evaluation.

2.2 Importance of Confinement

The presence of confinement is a central factor for the effect of ASR on structural performance. Reinforcing steel, loads on the concrete structure (e.g., deadweight), and the configuration of the structure (i.e., restraint offered by the structural layout) provide confinement that restrains in-situ expansion of the ASR gel and limits the resulting cracking in concrete. Structural testing of full-scale specimens simulates the in-situ confinement and therefore provides much more representative results than simpler approaches that do not account for confinement (e.g., material property testing).

Confinement limits ASR expansion of the in-situ structure, which reduces the extent of deleterious cracking and the resultant decrease in structural performance. Publicly available test data for structural performance of ASR-affected structures indicate a significant difference in results when adequate confinement is present. As an example, test data show that the one-way shear capacity of a specimen containing three-dimensional reinforcement was not significantly affected by ASR, but specimens without such reinforcement exhibited loss of capacity by up to 25% (References 1.4 & 1.5).

The difference in structural performance observed in published test data with varying degrees of confinement results from a "prestressing" effect. When reinforcement is present to restrain the tensile force exerted by ASR expansion, an equivalent compressive force develops in the concrete. If loads applied on the structure result in tensile stresses (direct, diagonal, or otherwise), the compressive stresses in the concrete must be completely overcome before additional tensile load is reacted by the reinforcement. Cracking in confined concrete would not occur until the tensile stress in the concrete exceeds the compressive stress in the concrete from the prestressing effect. The prestressing effect does not reduce the ultimate tensile capacity of the reinforcement. In some cases, literature indicates that the prestressing effect of ASR creates a stiffer structural component with a higher ultimate strength than an unaffected member[3]. Test data show that this prestressing effect applies even when ASR expansion has yielded the reinforcing bars. (Reference 1.5)

Given the interplay between ASR-induced cracking and structural restraint, it is imperative that evaluation of the structural impacts due to ASR focus on structural testing rather than material property testing of cores removed from the structure. The concrete prestressing effect is only present when the expansion is confined. If the concrete is removed from the stress field, the concrete prestressing effect is lost. A core sample from an ASR-affected, reinforced concrete structure will not be confined by the stresses imparted by the reinforcement and surrounding concrete after it is removed from the structure. Therefore, such a core is not representative of the concrete within its structural context. Measured mechanical properties from a core taken from a confined ASR-affected structure have limited applicability to in-situ performance; such results only represent the performance of an unconfined or unreinforced structure.

Figure 2-1 illustrates the effect of confinement with photographs of two surfaces of the same ASR-affected, reinforced concrete beam[4].

Confined Face of ASR-affected Beam (left); Unconfined face of Same ASR-affected Beam (right)

Figure 2-1. Effect of Confinement on ASR-affected Concrete

Based on the importance of the prestressing effect on structural performance, the typical approach of re-evaluating structural calculations using updated material properties from cores

[3] The planned approach for structural evaluations at Seabrook Station (MPR-4288) does not credit the possibility that ASR could increase the ultimate strength of the member in question.

[4] The beams shown in Figure 2-1 are not from the MPR/FSEL large-scale test programs.

would not be representative of structures at Seabrook Station. Instead, evaluations need to rely on structural test data of ASR-affected reinforced concrete.

2.3 Available Structural Test Data

The interim structural assessment considered the various limit states for reinforced concrete (e.g., shear, reinforcement anchorage) and applied capacity reduction factors based on data in publicly available literature. However, determination of appropriate reduction factors was limited by the poor representativeness of available data for ASR-affected concrete with reinforcement comparable to structures at Seabrook Station (i.e., two-dimensional reinforcement mats).

2.3.1 Shear Capacity

The interim structural assessment (Reference 2.1) assumed a strength reduction of 25% for out-of-plane shear (References 1.4 & 1.6), but this was a conservative treatment that is not necessarily representative of the expected performance of the walls at Seabrook Station.

- The available data on out-of-plane shear show a range of impacts from a reduction of 25% to a gain of 12% (Reference 1.4). Use of the 25% reduction for a structural assessment is on the conservative edge of the range.

- The shear capacity reduction due to ASR of 25% is based on small-scale testing using 5-inch × 3-inch beams (Reference 1.6). It is well known that shear test results do not scale well. In fact, the study that generated the results suggesting a 25% reduction specifically noted that the small test specimens likely exaggerated the deleterious effect of ASR, because the depth of ASR cracks is relatively greater in smaller specimens.

The literature review (Reference 2.2) included published research on large-scale testing, such as the research that had been performed at the Delft University of Technology on test specimens that had been recovered from an existing bridge deck that exhibited ASR (Reference 1.8). MPR concluded that these tests were less representative than the smaller scale laboratory tests discussed above. In the example of the Delft University study, test specimens included significant differences in configuration relative to structures at Seabrook Station. Specifically, the bridge deck had plain reinforcement (i.e., no deformation) with a low yield strength (approximately 30 ksi) and the specimens required extensive laboratory retrofit to generate a shear failure. In addition, the process of harvesting a specimen from an existing structure inherently results in damage that affects the results (see Section 2.4.1 for additional discussion).

2.3.2 Reinforcement Anchorage

The interim structural assessment (Reference 2.1) assumed a strength reduction of 40% for reinforcement lap splices in ASR-affected concrete (Reference 1.9), but this was a conservative treatment that is not necessarily representative of the expected performance at Seabrook Station.

- While the study producing an average strength reduction of 40% was the most relevant for the reinforcement anchorage limit state without transverse reinforcement, this study was based on a rebar pullout test method that is outdated and known to be unrealistic. In a rebar pullout test, the rebar is placed in tension and the concrete is placed in compression. This stress state is much different than the service condition for most reinforced concrete members, in which both the rebar and the surrounding concrete are in tension. Accordingly, a report from the ACI Technical Committee 408 stated that the rebar pullout method is "inappropriate and not recommended." (Reference 1.10)

- Testing performed for the study showing a 40% strength reduction used reinforcing steel significantly smaller (#5 bars) than the reinforcement in structures at Seabrook Station (typically #8 bars or larger for safety-related structures).

2.3.3 Anchor Capacity

Review of publicly available literature did not identify test data on capacity of anchors or shallow embedments in ASR-affected concrete (Reference 2.2).

For the interim structural assessment, MPR conducted testing on an ASR-affected bridge girder to provide a basis for the potential degradation.

2.3.4 Conclusion

While the literature review and girder testing provided information to support the interim structural assessment, it also highlighted that the state of knowledge on ASR did not include test data that were closely representative of reinforced concrete structures at Seabrook Station. Therefore, NextEra commissioned MPR to conduct testing to provide more representative data that would support follow-up structural evaluations.

2.4 Test Program Considerations

2.4.1 Test Specimen Approach

Large-scale structural testing of ASR-affected concrete typically involves specimens that are either harvested from existing ASR-affected structures or fabricated using constituents that accelerate ASR development. Table 2-1 summarizes the differences between these approaches.

Table 2-1. Comparison of Test Specimen Approaches

Harvested Specimens	Fabricated Specimens
Advantages • ASR developed along a timescale that represents an actual structure • Does not require capability to fabricate specimens and store specimens while ASR is developing	Advantages • Allows precise control of test variables, which permits separate effects testing where there is only one variable (e.g., ASR level) • Enables aging beyond currently-exhibited ASR levels • Common basis for ACI Code provisions
Disadvantages • The harvesting process may damage the test specimens and affect results • Range of testing is limited by currently-exhibited ASR levels	Disadvantages • ASR development is much faster than for actual structures

Specimens for the MPR/FSEL test programs were fabricated by FSEL so that the impact of ASR could be determined as a function of its severity, including levels of ASR expansion beyond those currently seen at Seabrook Station. The fabricated test specimens were designed with a reinforcement configuration and concrete mixture that represented structures at Seabrook Station to the maximum extent practical.

Using fabricated test specimens avoids the process of cutting out a section of reinforced concrete and transporting it to the laboratory, which results in damage that affects the test results. Specifically, the newly cut concrete surfaces would be subject to rapid expansion due to stress relaxation in the absence of the structural context. Additionally, cutting of rebar precludes its full development under loading, which also reduces representativeness. Design features of fabricated test specimens [redacted] can restore a portion of the continuity that represents the original structure, thereby making the test results more representative of true structural performance. For these reasons, published research using harvested test specimens (e.g., the Delft University study, Reference 1.8) was avoided, and structural tests relied primarily on fabricated specimens.

NextEra and MPR considered harvesting samples from the canceled Unit 2 at Seabrook Station, but ultimately decided against this approach. In addition to the damage incurred during the harvesting process, samples from Unit 2 would only be able to represent ASR-affected concrete to currently-observed expansion levels at Unit 2. Accelerated aging was an essential element of the MPR/FSEL test programs, because the results needed to address ASR-induced expansion that could occur in the future.

2.4.2 Representativeness Objectives of Test Programs

MPR designed test programs for NextEra to evaluate shear capacity, reinforcement anchorage, and anchor capacity with the following key features:

- Large size to represent the scale of structures at Seabrook Station
- Experimental design that is consistent with the design basis of Seabrook Station and accepted in the concrete industry
  - Test methods and experimental setups for shear and reinforcement anchorage testing are consistent with those used for tests that calibrate ACI Code equations
  - Test methods for anchor capacity testing are consistent with those performed in response to NRC IE Bulletin 79-02 (Reference 2.3)
- Specimen design that uses a reinforcement configuration and concrete mixture design that reflects reinforced concrete structures at Seabrook Station
- Presence of ASR to an extent that is consistent with levels currently observed at Seabrook Station and at levels that could be observed in the future

Additional details on these features are provided in the subsequent sections of this report.

Figure 2-2 presents various sources of information and indicates their relative representativeness for evaluating structural performance of ASR-affected reinforced concrete structures at Seabrook Station. The data set obtained as part of the MPR/FSEL test programs is a marked advancement from the collection of published literature sources and forms the definitive technical basis for evaluation of reinforced concrete structures at Seabrook Station for the applicable limit states.

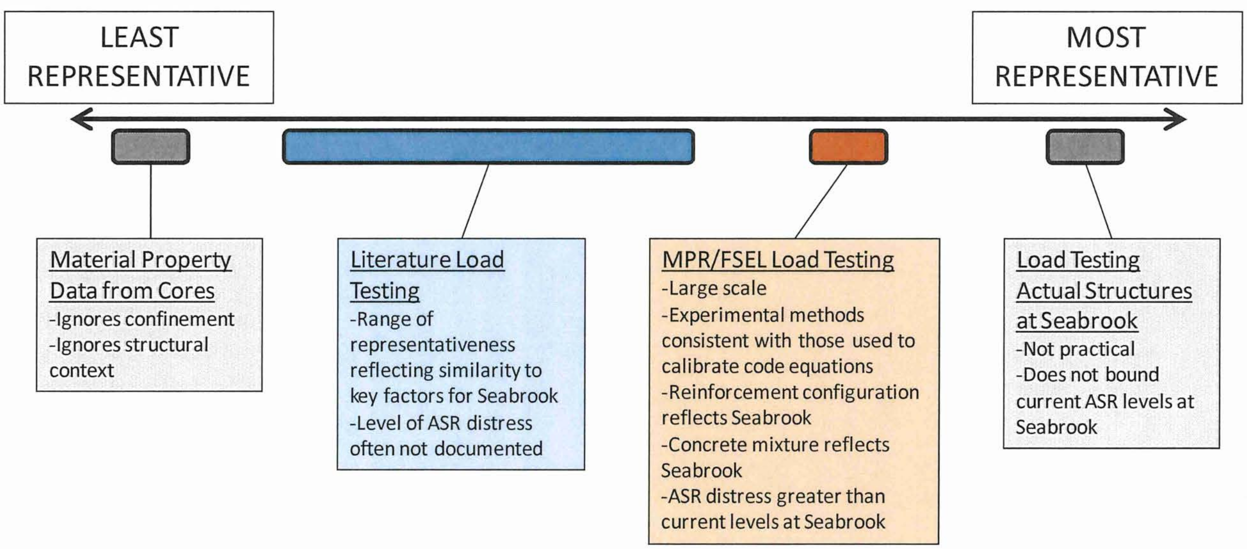

Figure 2-2. Representativeness of Information Sources for Evaluating Structural Performance

3 Test Specimen Configuration

Development of ASR in concrete and symptoms of ASR that can be used to monitor the condition of the concrete are strongly influenced by the design of the affected member. The MPR/FSEL test programs used specimens that represented reinforced concrete structures at Seabrook Station to the greatest extent practical. Fabricated test specimens were designed to incorporate specific features to maximize representativeness, while the bridge girder was selected for anchor testing because it contained high levels of ASR distress. Content in this section is drawn from References 3.3, 4.1, 4.2, and 4.3.

3.1 FABRICATED TEST SPECIMENS

3.1.1 General Description

Test specimens designed and fabricated for the test programs incorporated several key characteristics that provide strong representativeness to Seabrook Station, as follows:

- Reinforcement configuration of two-dimensional rebar mats with comparable reinforcement ratios to the plant in each in-plane direction
- Clear cover above reinforcement mats consistent with the plant. For the Shear, Reinforcement Anchorage, and Instrumentation Test Programs, the specimen design specified cover of 2 inches on the side representing the interior surface and 3 inches on the side representing the exterior surface. For the Anchor Program, the specimen design specified clear cover of 2 inches on both sides, which enabled installation and testing of anchors on both sides of the test specimen. Anchors of interest at Seabrook Station are installed on interior surfaces, so the presence of 3 inches of cover on the opposite wall face to simulate the exterior surface was not necessary.
- [redacted]
- [redacted]
- [redacted]
- Large overall size (see Table 3-1 for dimensional summary)

The concrete mixture design for the fabricated test specimens included highly reactive fine aggregate [redacted], which accelerated development of ASR. The shear, reinforcement anchorage, and instrumentation specimens also included reactive coarse aggregate and cement with high alkali content. In this manner, the test specimens could reach levels of ASR beyond that observed at Seabrook Station after only a short time of conditioning (i.e., maximum of 2.5 years for these test programs).

To the extent practical, concrete constituents were obtained from sources that were consistent with concrete at Seabrook Station. [redacted]

3.1.2 Differences between Specimens

The different purposes of the MPR/FSEL test programs necessitated dimensional differences between the fabricated test specimens. Table 3-1 below summarizes selected parameters of interest and the associated differences. Appendix A contains photographs, diagrams, and drawings of the test specimens.

Table 3-1. Comparison of Fabricated Test Specimens

Parameter	Anchor Block Specimens	Reinforcement Anchorage Specimens	24-inch Shear Specimens	Instrument Specimen
Height	[redacted]	[redacted]	[redacted]	[redacted]
Width	[redacted]	[redacted]	[redacted]	[redacted]
Length	[redacted]	[redacted]	[redacted]	[redacted]
Presence of Lap Splice	No	Yes	No	No
Vertical Rebar Size & Spacing	[redacted]	[redacted]	[redacted]	[redacted]
Horizontal Rebar Size & Spacing	[redacted]	[redacted]	[redacted]	[redacted]
Stirrups Size & Spacing	[redacted]	[redacted]	[redacted]	[redacted]

*Two half-length specimens were fabricated in a single placement

The most significant difference in the specimen configuration relates to the reinforcement ratio in the horizontal direction for the shear specimens. This difference was needed for two reasons: (1) for consistency with the shear test specimens used to derive the concrete contribution to shear strength for the design code and (2) to preclude failure of the test specimen via flexure at loads less than the expected shear capacity. The differences in reinforcement enabled a review of the potential impact of reinforcement ratio on ASR distress level and expansion behavior.

The anchor, shear, and reinforcement anchorage test specimens included transverse reinforcement (i.e., stirrups) outside of the test region to ensure that the test specimen failed in the test region by the desired failure mode. These stirrups also supported constructability. The differences in stirrup configuration enabled a review of the potential impact of confinement at the edges of the specimen on ASR distress and expansion behavior.

3.2 GIRDER TEST SPECIMENS

In addition to the fabricated test specimens, the Anchor Test Program also included testing on ASR-affected bridge girders. These specimens exhibited high levels of in-plane expansion, beyond what was achieved in the fabricated specimens. A bridge girder was used in the initial phase of the Anchor Test Program because it was available for immediate testing, which was necessary to support the interim structural assessment. A second phase of anchor testing used another bridge girder to obtain more test data at higher levels of expansion. The girder contains vertical #4 reinforcing bars spaced at 18 inches with a 1-inch minimum cover. Horizontal prestressing strands are also present at the bottom of the beam.

4 Characterizing ASR Development

The objective of each structural test program was to develop a trend for structural capacity as a function of ASR distress level. Accordingly, it was essential to accurately characterize the extent of ASR development in the test specimens. Routine monitoring of ASR development allowed load tests to be performed at pre-defined levels across the range of ASR distress achieved over the duration of the test programs.

Over the course of routine monitoring, observations on ASR development and expansion behavior informed decision making on the test program and ultimately influenced recommended monitoring practices at Seabrook Station.

This section discusses the efforts from the test programs to characterize ASR development, insights gained from these efforts that affected the course of the test programs, and the implications of key conclusions for structural evaluations and long-term monitoring at Seabrook Station. Content in this section is drawn primarily from References 4.1, 4.2, and 4.3.

4.1 Methods for Determining ASR Development

Several different methods were used to characterize ASR development in the fabricated test specimens:

- Expansion Monitoring - ASR-related expansion is a volumetric effect that results in dimensional changes in all three directions. FSEL monitored expansion on the surfaces adjacent to the reinforcement mats (i.e., the in-plane direction) and in the direction normal to the reinforcement mats (i.e., the through-thickness direction) using several different methods, including crack width summation, measurement of through-specimen embedded rods, and profiling of the specimen thickness in several locations over the specimen height.

- Material Properties - Technical literature identifies that ASR degrades the material properties of the concrete. FSEL tested concrete cylinders fabricated at the same time as the test specimens and cores obtained from the test specimens for compressive strength, elastic modulus, and tensile strength to quantify this degradation.

- Petrography - ASR distress may also be characterized by quantifying observed degradation symptoms in concrete samples. A petrographic examination was performed on a polished sample from a core taken from each test specimen at the time of load testing.
  The petrographer examined the sample under a microscope to confirm the presence of ASR and to quantify the extent of degradation using the Damage Rating Index (DRI) and Visual Assessment Rating (VAR) methodologies.

For the girder specimens used in the Anchor Test Program, FSEL performed in-plane expansion measurements prior to testing and provided a core to a petrographer to confirm the presence of ASR by petrographic examination[5].

4.2 Expansion Monitoring

4.2.1 Expansion Direction

All test specimens exhibited significantly more pronounced expansion in the through-thickness direction than the in-plane direction. Expansion in the in-plane direction plateaued at low levels, while expansion in the through-thickness direction continued to increase. Figure 4-1 is a plot of expansion for Specimen [redacted] and illustrates this behavior. Expansion behavior in this test specimen is typical of other fabricated test specimens[6].

The blue line represents expansion in the through-thickness direction. FSEL obtained most of these measurements from pins that were embedded in the test specimen during fabrication (open data points). In May 2015, FSEL implemented a more comprehensive approach whereby thickness measurements along the height profile of the specimen were averaged (solid data points). The red and green lines represent expansion in the in-plane directions (horizontal and vertical) obtained using embedded pins. The orange line represents expansion in the in-plane directions from crack width measurement (i.e., cracking index).

[5] DRI and VAR were not utilized on the girder cores.

[6] Expansion of the girder specimens from the Anchor Program was measured at the time of testing, but was not monitored with time. The instrumentation specimen exhibited comparable in-plane expansion, but through-thickness expansion was strongly influenced by the lack of stirrups on the beam ends (see Section 4.2.5).

[redacted]

Figure 4-1. ASR-related Expansion in Specimen [redacted]

At low expansion levels ([redacted]% to [redacted]2%), expansion occurred in all three directions. At higher ASR levels, expansion occurred preferentially in the through-thickness direction.

The difference between in-plane expansion and through-thickness expansion is due to reinforcement detailing and the resulting difference in confinement between the in-plane and through-thickness directions. The reinforcement mats confine expansion in the in-plane directions, whereas the lack of reinforcement in the through-thickness direction allows free expansion. Therefore, expansion occurs preferentially in the through-thickness direction.

4.2.2 Assessment of Combined Cracking Index Methodology

NextEra has been monitoring expansion of ASR-affected concrete at Seabrook Station using crack width measurement (i.e., combined cracking index (CCI)) since 2011. Measurement of concrete expansion can be approximated by crack width summation because concrete has minimal capacity for expansion before cracking. While true engineering strain is represented by the sum of material elongation and crack widths, the crack width term rapidly dominates the overall expansion.

As shown in Figure 4-1, in-plane CCI values agreed closely with the observed expansion from embedded pins in terms of both the trend and magnitude. The expansion values measured using embedded pins are a better measure of true engineering strain because these measurements reflect both material elongation and crack width. However, because of the close agreement with CCI, results from the MPR/FSEL test programs for expansion monitoring support use of CCI as an approximation for in-plane expansion.

The procedure used by FSEL personnel to determine CCI was controlled under the FSEL Quality Assurance program and was identical to the procedure used to determine CCI at Seabrook

Station. To assess the repeatability of CCI measurements obtained by FSEL personnel, the individual performing CCI at Seabrook Station traveled to FSEL to perform measurements on the test specimens (Reference 2.4). In general, results from this effort were consistent with results obtained by FSEL personnel with an average difference of [redacted] mm/m. For most locations, the results were very close. The most significant difference in the measurements was related to the minimum recording threshold for a crack width. The Seabrook methodology only includes cracks with a width of 0.05 mm/m or greater. Evaluation of the CCI comparison results indicated that different operator judgment of the width of very small cracks resulted in the different CCI values. Where ASR is more significant, cracks are larger and repeatability improves. The threshold for structural evaluations at Seabrook Station is 1.0 mm/m, so measurement variability in the range observed by the CCI comparison study is acceptable.

An important advantage of the CCI methodology for Seabrook Station is that results can be used to approximate total expansion in the in-plane directions since the time of original construction. Other methodologies (e.g., installing reference pins and monitoring change in relative position) only determine expansion since the time of the first measurement, which establishes the baseline.

4.2.3 Large Crack on Specimen Edge

As ASR developed in the test specimens, a large crack was noted in the center of the surfaces of the beam that were between the reinforcement mats. Figure 4-2 is a photograph showing the large crack in one of the beam specimens.

[redacted]

Figure 4-2. Large Crack from Surface between Reinforcement Mats

This large crack is not representative of expansion behavior of structures at Seabrook Station, which have a network of members that are either cast together or integrally cast with special joint reinforcing details. In an actual structure, a vertical wall with two-dimensional reinforcement will be confined in the through-thickness direction at its intersection with neighboring members (i.e., at the top and bottom with floor and ceiling slabs, at the sides with perpendicular walls, and uniformly along the wall face by the subgrade for below grade external walls). The confinement provided by the network of members in a structure is likely sufficient to preclude large cracks like those seen in the FSEL test specimens.

Sectioning of Test Specimens

To confirm that this large crack was an edge effect that did not compromise the representativeness of the test region, FSEL sectioned the beam cross section (i.e., cut with a saw) to assess the depth of the crack for one anchor test specimen and two shear test specimens (after testing was completed). In all cases, FSEL observed that the large crack penetrated only a few inches into the specimen height.

Although the large crack was an edge effect, it was not clear whether it had affected the ability to measure expansion in the through-thickness direction using the embedded pins (which are shown in Figure 4-2). The large crack concentrated the expansion between the embedded pins, rather than distributing the expansion across the entire specimen width, as would be expected in actual structures at Seabrook Station. Damage incurred to the specimens by the sectioning process and the immediate expansion after sawing resulting from relaxation of confinement prevented quantitative evaluation of the sectioned specimen.

Expansion Measurements over Specimen Height Profile

FSEL developed a new methodology for measuring expansion in the test specimens that obtained measurements along the entire height of the shear and reinforcement anchorage test specimens using a laboratory-fabricated frame (i.e., the z-frame). The frame fit around a test specimen and enabled repeatable measurements of through-thickness (i.e., z-direction) expansion at nine points along the height of the beam. Figure 4-3 provides a plot showing the expansion profile for Specimen [redacted] using the nine measurement locations. The blue dots and solid line show the nine specific points and the dashed line gives the average value. This plot is typical of the other test specimens.

[redacted]

Figure 4-3. Expansion Profile of Specimen [redacted] (as Measured with the Z-Frame)

The z-frame expansion measurements demonstrated that the expansion measured near the edge of the beam (i.e., where the large crack exists) is consistent with the expansion measured over the entire beam height. Based on the relatively low variation about the mean, the results of the z-frame expansion study confirmed that use of an average value to describe through-thickness expansion of the entire specimen is appropriate.

Crack Development Profile

The z-frame data and the observations from sectioning indicate that while total expansion in the through-thickness direction is consistent across the profile of the test specimen, the cracking behavior is different. These observations suggest that along the specimen edges, expansion is concentrated into a large crack; whereas away from the edges, expansion is distributed into finer cracks along the specimen cross-section. Figure 4-4 illustrates this expansion behavior.

[redacted]

Figure 4-4. Expansion Behavior of Test Specimens

4.2.4 Effect of Reinforcement Ratio on Expansion

Test specimens from all test programs exhibited comparable expansion behavior in the reinforced (i.e., in-plane) directions. The magnitude of ASR-related expansion in each case plateaued at ~[redacted] to [redacted]%. These observations indicate that the differences in reinforcement ratio between the shear test specimens ([redacted]%), the reinforcement anchorage and instrumentation test specimens ([redacted]%), and the anchor test specimens ([redacted]%), did not have a noticeable effect on the expansion behavior of the test specimens. The nature and magnitude of ASR-related expansion is more affected by the direction of the reinforcement than the reinforcement ratio. The test specimens were reinforced in the same direction, and as a result, experienced similar directionality in ASR-related expansion.

4.2.5 Effect of Stirrups at Ends of Specimen on Expansion

Expansion monitoring from the various test specimens identified that the presence of any level of confinement at the specimen ends was an important parameter for expansion behavior. Fabricated specimens for the Shear, Reinforcement Anchorage, and Anchor Test Programs included stirrups (ranging from [redacted] to [redacted] stirrups) on each end of the beam. Development of ASR in the through-thickness direction was comparable for these specimens (up to ~[redacted]% maximum over ~2.5 years; all values obtained away from the stirrup region).

The instrumentation specimen did not include stirrups on the end of the specimen and the resulting expansion caused a wide crack in the concrete between the reinforcement mats. Measured through-thickness expansion at the ends of the beam exceeded [redacted]% after one year. The wide crack in the instrumentation specimen was an exaggerated version of the mid-plane crack described in Section 4.2.3; however, this crack progressed from the end of the specimen toward the center, where expansion was less than [redacted]% after one year. The ends of concrete members at Seabrook Station have some confinement in the through-thickness direction (e.g., connection with a wall). Accordingly, the expansion behavior of the shear, reinforcement anchorage, and anchor test specimens is more representative of the plant.

4.2.6 Environmental Conditioning Effects

ASR proceeds more rapidly in hot and moist conditions. Test specimens were stored in an Environmental Conditioning Facility (ECF) with alternating wet and dry cycles to promote ASR development. To simulate the potential presence of groundwater on one side of the reinforced concrete at Seabrook Station, FSEL wetted absorbent fabric that was placed on the top side of each specimen. Misters in the ECF maintained a humid environment during wet cycles.

Comparison of expansion data from both sides of the test specimens did not identify a discernible bias in ASR development resulting from the wet fabric. The internal humidity of the concrete and the atmospheric conditions in the ECF were sufficient to drive progression of ASR uniformly throughout the test specimens.

4.2.7 Additional Testing - Confined Cubes

FSEL is currently performing a study to monitor expansion of a set of 19-inch cubes with varying reinforcement configurations and concrete mix designs. A total of 33 cubes are involved in the study. This testing is not part of the MPR/FSEL test programs for NextEra, but does provide valuable insights on expansion behavior.

Preliminary results indicate that the most significant factor for expansion behavior is the presence of reinforcement or lack thereof (Reference 6.1). Specific observations include the following:

- Cubes with one-dimensional reinforcement exhibited significantly less expansion in the reinforced direction than the unreinforced directions. Variation of the reinforcement ratio in the reinforced direction did not affect the relative degree of expansion in any direction. The same relative distribution of expansion was observed for cubes with two-dimensional reinforcement. This expansion behavior is consistent with the results from the MPR/FSEL test programs, where expansion occurred predominantly in the unreinforced direction.

- Cubes with unequal two-dimensional and three-dimensional reinforcement exhibited slightly less expansion in the directions with higher reinforcement ratios. Specifically, a reinforcement ratio difference of 1.1% vs. 0.5% resulted in a maximum expansion differential of about 0.1% between the different directions. These results are consistent with the conclusion from the MPR/FSEL test programs that differences in reinforcement ratio between the various types of test specimens did not have a noticeable effect on the aging mechanism.

- Cubes with identical reinforcement configurations, but slightly different concrete mix designs (i.e., substitution of coarse aggregate that is not reactive) resulted in comparable expansion behavior in terms of the relative distribution of expansion in the different directions. While the specimens for each MPR/FSEL program used a common concrete mix design, all specimens came from different batches with minor variations. The repeatable results among the MPR/FSEL program test specimens are consistent with the observation from the new FSEL expansion study, that the presence (or lack) of reinforcement is more impactful than minor differences in the concrete mixture (as would be expected with different concrete placements during original construction of Seabrook Station).

4.2.8 Comparison to Literature

The expansion behavior of the test specimens agrees with literature data from many sources, as summarized in References 1.2, 1.3, and 2.2. Of particular interest is Reference 1.11, which reports on ASR expansion of concrete blocks with varying reinforcement. This study concluded that the presence of reinforcement decreased the expansion parallel to the reinforced direction, without reducing (and in some cases increasing) expansion in other directions. Literature sources state that dominant cracks form parallel to the direction of reinforcement, which is consistent with the observation from the MPR/FSEL test programs that the majority of the expansion occurred in the through-thickness (i.e., the unreinforced) direction. Additionally, the literature sources are consistent with the observation of the large crack between the reinforcement mats observed in the test specimens for the MPR/FSEL test programs.

Data collated from multiple studies in Reference 1.2 yielded a conclusion that even a comparatively small amount of reinforcement significantly restrains expansion. This conclusion supports the observation on the effect of stirrups, which significantly reduced expansion in the regions of the beams where they were present.

4.3 MATERIAL PROPERTIES

In addition to expansion monitoring, concrete material properties of the test specimens were used as an independent means for monitoring progression of ASR. To determine the baseline, FSEL tested cylinders that were fabricated at the same time as the test specimens. To determine the ASR-affected material property, FSEL obtained and tested cores from each specimen at the time of testing. For the instrumentation specimen, FSEL tested cores that were removed as part of instrument installation.

Test Results

For the shear, reinforcement anchorage, and instrumentation test specimens, FSEL performed material property testing for compressive strength and elastic modulus. Results were normalized by calculating the ratio of the material property at the time the core was obtained to the material property result from the corresponding 28-day cylinder. Figures 4-5 and 4-6 present the material properties as a function of through-thickness expansion for the reinforcement anchorage test specimens (A-Series; blue diamonds), shear test specimens (S-Series, green triangles), and instrumentation specimen (IB-Series; purple circles).

[redacted]

Figure 4-5. Normalized Compressive Strength of Test Specimens

[redacted]

Figure 4-6. Normalized Elastic Modulus of Test Specimens

Figure 4-5 indicates a relatively shallow decrease in compressive strength as a function of ASR development, which is consistent with literature data. As compared to compressive strength, modulus of elasticity (Figure 4-6) exhibited a greater sensitivity to ASR-related degradation and less data scatter. The observation that elastic modulus is a stronger function of expansion is consistent with literature.

Although FSEL performed compressive strength testing on cylinders and cores representing anchor test specimens, these data are not included in Figure 4-5. The methodology for determining through-thickness expansion of the block anchor test specimens was less sophisticated, so direct comparison of the results with those from the shear, reinforcement anchorage, and instrument specimens is somewhat misleading. The material property test data from the anchor test specimens show average normalized compressive strengths of approximately [redacted] and [redacted] at through-thickness expansions of about [redacted]% and [redacted]%, respectively. These data agree with the overall conclusion of a relatively shallow decrease as a function of ASR development. Through-thickness measurements from the girder series anchor tests were not possible, so compressive strength data cannot be directly compared with the other results. Elastic modulus results were not obtained as part of the Anchor Test Program, so anchor test specimen data could not be included in Figure 4-6.

As part of the Shear, Reinforcement Anchorage, and Instrumentation Test Programs, FSEL also performed testing on cylinders and cores for splitting tensile strength, although this practice was instituted late in the MPR/FSEL test programs, so only limited data are available. These data showed a weak sensitivity to ASR development.

Comparison of Material Property Data for Different Test Programs
As identified in published literature (e.g., Reference 1.2), changes in material properties are characteristic of the ASR aging mechanism. The results observed in the MPR/FSEL test programs identify no discernible difference between the test specimens over the course of aging, despite the differences in dimensions, reinforcement ratios, and presence of stirrups between the various specimens. The consistent relationship between aging and expansion for the various beam designs suggests that the aging mechanism is insensitive to the specific boundary conditions of a particular specimen design.

4.4 PETROGRAPHY

4.4.1 Presence of ASR

Cores were obtained from most test specimens for petrographic examinations, which were performed by Wiss, Janney, Elstner Associates (WJE) to assess the general properties of the concrete and to confirm the presence of ASR.

The results of the petrographic investigations confirmed the presence of ASR in the test specimens and determined that results of ASR were observed throughout the entire test specimen, not just at the surface. For cores from the control specimens, petrographic examinations noted the presence of ASR gel in pores and voids, but there were no indications of concrete distress. Therefore, the control specimens provided an appropriate baseline for the test programs.

4.4.2 Investigation of Petrography as a Correlating Parameter

For shear and reinforcement anchorage specimens, WJE also determined the degree of ASR using Damage Rating Index (DRI) and Visual Assessment Rating (VAR). Both methods rely on tabulating visual observations to quantify the extent of ASR distress. The DRI and VAR methods have been used in evaluation of cores from Seabrook Station. Petrographic studies were included in the test programs to determine if Traditional DRI, Modified DRI (which incorporates symptoms of ASR in fine aggregate), or VAR could be used to estimate expansion to-date at Seabrook Station.

Figures 4-7 and 4-8 compare the petrographic examination results against the corresponding through-thickness expansion for each test specimen.

[redacted]

Figure 4-7. DRI (Traditional and Modified) vs. Through Thickness Expansion

[redacted]

Figure 4-8. VAR vs. Through Thickness Expansion

When compared to measured through-thickness expansion, Traditional DRI, Modified DRI, and VAR all increased as ASR degradation increased. However, the scatter in the data increased at higher levels of ASR-related expansion. In addition, interpretation of petrographic examination results depends on petrographer judgment, which is less repeatable than purely quantitative measurements. Therefore, it may be misleading to apply a correlation of DRI or VAR to through-thickness expansion based on measurements made by another petrographer, such as those of concrete cores from Seabrook Station. Accordingly, MPR does not recommend using DRI or VAR to correlate expansion levels in the test programs with those at Seabrook Station.

4.5 CONCLUSIONS

As part of the MPR/FSEL test programs, MPR evaluated test data for ASR development across the various specimen types. Key conclusions from an evaluation of all data include the following:

- Observed expansion in the test specimens was much greater in the through-thickness direction than in the in-plane directions. The test specimen design included two-dimensional reinforcement mats that confined expansion in the in-plane directions, which is representative of Seabrook Station. These observations are consistent with published literature, which indicates that expansion of reinforced concrete will occur predominately in the unreinforced direction(s).

- The rate of expansion was approximately the same in all three directions until expansion reached [redacted]% - [redacted]% (i.e., [redacted] mm/m). In-plane monitoring by crack width summation (i.e., CCI) sufficiently characterizes ASR development until this level, after which through-thickness monitoring is required to track further ASR expansion.

- Total expansion in the through-thickness direction is consistent across the profile of the test specimen. However, the cracking behavior is different. At the test specimen edges, expansion is concentrated in a large crack that runs the length of the surface; whereas away from the edges, expansion is distributed into finer cracks across the test specimen cross-section. The single large crack is an edge effect and is not representative of structures at Seabrook Station.

- CCI values agree closely with the observed in-plane expansion from embedded pins, which is more representative of true strain. Based on this close agreement, CCI data obtained by Seabrook Station is confirmed to be a reasonable approximation for in-plane expansion. Additionally, a study of CCI measurements performed by FSEL personnel and the individual performing CCI for NextEra at Seabrook Station confirmed that repeatability is suitable for monitoring expansion at Seabrook. The procedure used by FSEL is the same as the procedure used at Seabrook.

- The internal humidity of the concrete and the atmospheric conditions in the ECF were sufficient to drive progression of ASR uniformly throughout the test specimens. Wet fabric placed on the top side of the test specimens to simulate groundwater at Seabrook Station did not result in a discernible bias in ASR development.

- Material properties decreased with increasing ASR-related expansion. Elastic modulus was the property that was most sensitive to ASR degradation. The trend between elastic modulus and ASR expansion was also the most repeatable among the material properties investigated. Therefore, elastic modulus is preferred over compressive strength or splitting tensile strength as a parameter for determining ASR development in the absence of monitoring instrumentation.

- The consistent relationship between material properties and expansion for the various beam designs suggests that the specific boundary conditions of a particular specimen design do not affect the ASR aging mechanism.

- Petrographic investigation of cores obtained at the time of testing confirmed the presence of ASR. Cores from control specimens showed ASR gel, but only in voids, and without accompanying concrete distress, which established that the control specimens were free of ASR degradation. Quantitative petrographic results using DRI and VAR trended with observed through-thickness expansion measurements. However, the data scatter increased significantly at higher levels of ASR distress. In addition, the DRI and VAR methodologies rely on subjective petrographer judgment and may not be as repeatable as more purely quantitative methods. Accordingly, neither technique is recommended for correlating expansion levels in the test programs with those at Seabrook Station.

5 Test Results

Testing performed at FSEL included four test programs completed during a period of about four years. The test reports for the test programs provide detailed results (References 4.1, 4.2, & 4.3). This section summarizes the results from each test program.

5.1 Anchor Testing

The purpose of the Anchor Test Program was to quantify the relative impact of ASR on anchor performance by comparing anchor tests at various levels of ASR expansion to tests performed prior to the development of ASR.

5.1.1 Test Description

The approach for anchor testing was consistent with testing performed by the anchor vendor (Hilti) for original construction of Seabrook Station. The vendor testing was used as an input to the plant evaluation demonstrating compliance with NRC IE Bulletin 79-02, which represents the plant design basis for anchor bolts.

FSEL performed testing on two ASR-affected girders, and [redacted] fabricated test specimens that were designed to reflect reinforced concrete at Seabrook Station to the extent practical[7].

Two different types of anchors were used to represent post-installed anchors and cast-in-place embedments at Seabrook Station: the Hilti Kwik Bolt 3 expansion anchor, and the Drillco Maxi-Bolt undercut anchor.

- The Hilti Kwik Bolt 3 is the preferred torque-controlled expansion anchor for Seabrook Station. It is a more modern version of the Hilti Kwik Bolt 1 and Kwik Bolt 2 anchors that were used when Seabrook Station was constructed and installed over time at the beginning of plant life. The Kwik Bolt 3 is representative of its predecessors, as the basic design of the anchor family has not significantly changed.

- The Drillco Maxi-Bolt is an undercut anchor used at Seabrook Station. Undercut anchors are similar to cast-in-place anchors as they both utilize a positive bearing surface to transfer load to the concrete. Thus, undercut anchors are suitable representatives of cast-in-place anchors.

A range of anchor sizes and embedment depths were used for the series of tests. FSEL installed some anchors shortly after fabrication (i.e., prior to ASR development) and some anchors just

[7] FSEL fabricated [redacted] specimens, but one-specimen was not tested.

before testing (i.e., after ASR development). Anchors installed shortly after fabrication were set prior to ASR development, so expansion occurred around the anchor shank. Anchors installed just before testing were set after ASR development, so expansion was independent of the presence of an anchor. These conditions simulated the potential bounding conditions at Seabrook (i.e., anchor installed at original construction; anchor installed into ASR-affected concrete as part of a recent modification).

Anchor performance was evaluated using an unconfined tension test. This test method applies a tensile load to the anchor, and uses a reaction frame to distribute the load to a concrete surface a sufficient radius away from the anchor to avoid any confining stress (which could preclude concrete breakout). Load is increased until anchor failure, which occurred by one of the following modes:

- Concrete Breakout - Fracture of the concrete around the anchor in a cone-like shape emanating from the anchor head.
- Anchor Failure - Fracture of the anchor shank.
- Anchor Pull-out/Pull-through - Loss of load resistance due to local concrete failure and/or deformation of the anchor head. (This mode only applies to expansion anchors; i.e., the Hilti Kwik Bolt 3 for this test program.)

The level of ASR degradation was characterized by in-plane expansion, as measured using crack width summation (i.e., Combined Cracking Index). In-plane expansion due to ASR creates microcracks parallel to the axis of an anchor, which are most pronounced in the concrete cover. These microcracks that open perpendicular to the concrete surface have the potential to provide a preferential failure path within a potential breakout cone, leading to degraded anchor performance.

5.1.2 Test Results

Expansion Anchors

Figure 5-1 presents the results of unconfined tension testing of Hilti Kwik Bolt 3 expansion anchors in the girders and the blocks. Test results have been normalized relative to the measured 28-day compressive strength of the specimen, as failures were related to anchor pull-out/pull-through or concrete breakout (not anchor failure). Figure 5-1 includes results from the range of tested anchor sizes and embedment depths. For reference, the dashed lines show the theoretical concrete failure load for each anchor type, normalized by the measured 28-day compressive strength of the control test specimen, which was not affected by ASR.

[redacted]

Figure 5-1. Kwik Bolt 3 Anchor Test Results

The results presented in Figure 5-1 indicate that there is no performance reduction for expansion anchors when in-plane expansion is less than [redacted] mm/m, which is the maximum ASR level exhibited by the test specimens used for expansion anchor testing.

The majority of the test results were for in-plane expansion at [redacted] mm/m or less, because in-plane expansion of the block specimens did not exceed this level. The girder series tests extended the range of expansion covered by the test program. The low level of in-plane expansion in the fabricated specimens is consistent with the test specimens fabricated for the other test programs, which were also designed with two-dimensional reinforcement mats that provide confinement in the in-plane direction and closely represent the reinforced concrete at Seabrook Station.

Undercut Anchors

Figures 5-2 and 5-3 present the results of unconfined testing of Drillco Maxi-Bolt undercut anchors in the girders and the blocks. Results from the range of tested anchor sizes and embedment depths are provided. The dashed lines show the normalized theoretical concrete failure load for each anchor type.

Some of the Drillco Maxi-Bolt tests were installed at a depth less than the manufacturer's recommendation to ensure that tensile performance was limited by concrete failure, and would therefore investigate the effect of ASR in the concrete. Figure 5-2 provides the results of shallow depth testing. Test results in Figure 5-2 were normalized relative to measured 28-day compressive strength of the specimen, because anchor failure was related to concrete breakout. Figure 5-3 provides the results of full depth testing. Test results in Figure 5-3 were not normalized for compressive strength of concrete, because failure of full depth undercut anchors is governed by steel failure of the anchor (i.e., concrete strength is not limiting).

[redacted]

Figure 5-2. Shallow Drillco Maxi-Bolt Anchor Test Results

[redacted]

Figure 5-3. Full-Depth Drillco Maxi-Bolt Anchor Test Results

The results presented in Figures 5-2 and 5-3 indicate that no decrease in anchor performance was observed until in-plane expansion exceeded [redacted] mm/m. The reduction in performance observed in the test program was only for anchors installed at a significantly reduced embedment depth such that concrete failure limits anchor performance. Anchors with full embedment depth in ASR-affected concrete may perform satisfactorily at an expansion level of 8 mm/m or higher.

Anchor Installation Timing

Figures 5-1, 5-2, and 5-3 include results from testing of anchors installed shortly after specimen fabrication (i.e., before development of ASR) and anchors installed just prior to testing (i.e., after development of ASR). Test results indicate that there is no significant difference in anchor performance related to when the anchor was installed.

Through-Thickness Expansion

For the block specimens, through-thickness expansion was estimated at [redacted]% for [redacted] of the test specimens and [redacted]% for [redacted] specimens. The results indicate that anchor performance is not sensitive to through-thickness expansion.

Through-thickness expansion has the potential to create microcracks perpendicular to the axis of an anchor. These potential microcracks that open parallel to the concrete surface do not provide a preferential failure path to result in degraded anchor performance. An anchor loaded in tension would compress the through-thickness expansion and close any potential microcracks within the area of influence of that anchor. Without a "short-circuit" of the breakout cone, through-thickness expansion does not affect anchor performance. This observation with through-thickness expansion is in contrast to in-plane expansion where the potential for a "short-circuited" breakout cone exists.

5.1.3 Additional Testing - Confined Anchor Tests

During the first phase of the girder series in 2012, FSEL performed confined anchor testing that focused on the pullout behavior of expansion anchors in ASR-affected concrete. The testing rig for the confined tests placed the reaction load in the area immediately around the anchor, which prevents the breakout failure mode. The testing demonstrated that there is no significant loss of pullout/pull-through anchor capacity in ASR-affected concrete until higher levels of ASR expansion. Minor losses were observed beginning at an in-plane expansion of [redacted] mm/m.

The confined anchor test data were not included in the test results described in Section 5.1.2, because the stress state in the concrete around the anchor was not consistent with actual conditions for anchors in-service.

5.2 SHEAR TESTING

The purpose of the Shear Test Program was to determine the effect of ASR on out-of-plane shear capacity of reinforced concrete elements without shear reinforcement.

5.2.1 Test Description

The effects of ASR were evaluated using three-point bending tests on large reinforced concrete beams. [redacted] [redacted]-inch wide shear test specimens were fabricated for this test program. [redacted] of these specimens were controls that were tested approximately 30 days following fabrication (i.e., prior to the development of ASR). The other [redacted] test specimens were allowed to develop ASR and were evaluated relative to the performance of the control tests[8]. Figure 5-4 shows the test setup for the [redacted]-inch shear test specimens.

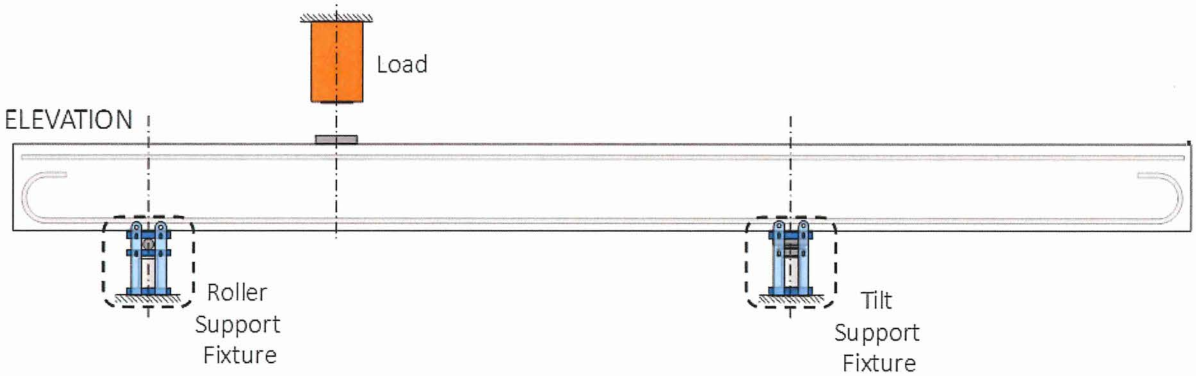

Figure 5-4. Test Setup for [redacted]-inch Shear Test Specimens (Elevation View)

The test span, or test region, is defined as the region between the point where the load is applied and the nearest support point. This loading configuration made it possible to conduct one shear test on each end of the shear test specimens, thereby providing two sets of test results for each specimen.

ACI 318 defines shear capacity based on the onset of diagonal cracking. During the load test, FSEL identified this point visually. In addition, the test equipment monitoring load as a function of deflection would indicate a slight reduction in load followed by a reduction in the slope of the overall response. Load testing continued until failure of the specimen, as identified by a rapid loss in load carrying capacity.

5.2.2 Test Results

Figure 5-5 provides the stress-displacement plots for the [redacted] shear test specimens. For clarity, only one of the two tests from each specimen is presented. The pair of results from each test specimen were nearly identical, so Figure 5-5 is representative of all [redacted] shear test results. The stress was normalized by the measured 28-day compressive strength of concrete for consistency with the approach used in ACI code calculations.

[8] Results from one of these test specimens ([redacted]) is for information only due to a test specimen nonconformance.

[redacted]

Figure 5-5. Normalized Shear Stress-Deflection Plots for [redacted]-inch Shear Test Specimens

The dashed circle indicates the region where diagonal cracking appeared, which is the shear capacity defined by ACI 318. The [redacted] plots in Figure 5-5 (representing twenty shear tests) indicate a clear and repeatable trend of higher levels of ASR expansion correlating with higher shear capacity. All [redacted] of the shear test results exceed the theoretical shear capacity calculated per ACI 318-71, which is a normalized shear capacity of 2.0. The apparent increase in shear capacity resulting from ASR is explained by the prestressing effect discussed in Section 2.2. The large number of tests and the repeatability of the data provide strong confidence in the conclusion that there was no adverse effect on shear capacity at the expansion levels tested.

5.2.3 Comparison to Literature

Published literature on structural testing of ASR-affected reinforced concrete includes a range of results that generally reflects the degree of reinforcement. Literature notes that triaxially reinforced concrete will only be slightly affected even by fairly severe ASR expansions (Reference 1.1). As discussed in Section 2.3.1 of this report, published literature of ASR-affected test specimens without shear reinforcement indicate shear capacity results ranging from a slight increase to a loss of 25%. Based on the results from the Shear Test Program showing no loss in shear capacity, the test specimens actually behaved more like triaxially reinforced concrete. Because the MPR/FSEL test program specimens were much more representative of Seabrook Station than published literature (e.g., [redacted] × [redacted] specimen cross-section, as compared to 5″ × 3″) and the MPR/FSEL test results were highly repeatable, structural evaluations for Seabrook Station can use the MPR/FSEL conclusion (i.e., no loss of capacity) in lieu of the results from published literature.

5.2.4 Additional Testing - [redacted]-Inch Specimen, Retrofits, and Uniform Loading

[redacted]-Inch Specimen

[redacted] inch test specimen was tested prior to the development of ASR to evaluate the effect of specimen depth on shear capacity. The specimen was designed and fabricated with reinforcement detailing typical of structures at Seabrook Station and a concrete mix design identical to the other shear test specimens. Although the allowable shear stress in the ACI code is independent of beam depth, there are test data that show the shear stress at initiation of diagonal cracking decreases at greater beam depths (Reference 1.7). The Shear Test Program included evaluation of the effect of specimen depth to ensure that it could be taken into account if tests of ASR-affected specimens had shown a decrease in shear capacity.

Results from this testing indicate that the normalized shear capacity of the [redacted]-inch test specimen was less than that observed in the [redacted]-inch control specimens. The normalized capacity was approximately [redacted]% of the theoretical value specified by the ACI code. This result is consistent with the data available in the ACI database for shear tests of larger width specimens (Reference 1.12). It is important to note that this test was conducted on a non-ASR-affected test specimen and does not impact the conclusions regarding the effect of ASR-related expansion on shear performance.

Retrofit Concept Testing

The original scope of the Shear Test Program included testing of retrofit concepts on specimens exhibiting ASR-induced expansion above which a deleterious effect was observed. A reduction in shear capacity was not observed at the highest expansion levels exhibited by the test specimens, so retrofit testing was not performed as part of the test program.

FSEL performed proof-of-concept testing on retrofit concepts installed in trial specimens (Reference 6.3). Shear performance of specimens with retrofits was compared to shear performance of control specimens. Two retrofit methods were investigated in this testing: (1) undercut anchors installed in the through thickness direction and tensioned on the surface with a nut and plate to provide confinement, and (2) threaded rod grouted into a drilled hole in the concrete and tensioned on the surface with a nut and plate. Four specimens were fabricated for this testing and each specimen was tested on both ends. Table 5-1 summarizes the test specimens used for retrofit testing.

Table 5-1. Proof-of-Concept Testing for Shear Retrofit

Specimen	End	Shear Reinforcement	Retrofit
LD1	North	No	None
LD1	South	No	None
SR1	North	No	Grouted Rods
SR1	South	No	Undercut Anchors
SR2	North	No	Undercut Anchors
SR2	South	No	None

Table 5-1. Proof-of-Concept Testing for Shear Retrofit

Specimen	End	Shear Reinforcement	Retrofit
SR3	North	Yes	None
SR3	South	No	Grouted Rods

Test results indicated that both undercut anchors and grouted rods were effective at shear strengthening. Shear strength and deformation capacity can be increased significantly by adding the retrofit anchors. The anchors behave similar to cast-in-place transverse reinforcement.

Uniform Load Testing

The test setup for the Shear Test Program used asymmetric three-point loading. Use of point loads is convenient and consistent with the test data used to calibrate the ACI code equations for shear. A uniform distribution would be more representative of the loads applied to some structures (e.g., hydrostatic loading on the exterior surface of a below-grade wall). Information in technical literature on the effect of uniform loading is generally based on small-scale test specimens, and indicates a higher capacity with uniform loading. FSEL performed uniform load shear testing on two sets of specimens with designs comparable to the specimens for the Shear Test Program. Force was applied using an air bladder to exert uniform pressure to the underside of each specimen. (References 6.4 & 6.5)

The first set of tests (Reference 6.4) included six beam specimens, three with point loading comparable to the Shear Test Program, and three with uniform loading applied over the middle 2/3 of the test specimen. For these tests, uniformly loaded specimens exhibited a slightly higher shear capacity than specimens subjected to point loads. Additional data on two 24-inch specimens were obtained as part of an investigation of uniform load testing of 48-inch specimens (Reference 6.5). For those tests, the uniformly loaded specimen exhibited lower shear capacity than the specimen subjected to point loads.

In the second set of tests (Reference 6.5), two 48-inch thick specimens and two 24-inch thick specimens were fabricated. The design of these specimens was comparable to the Shear Test Program specimens, although the 48-inch specimens were considerably longer (i.e., 45 feet, 4 inches). One specimen of each thickness was tested with uniform load and one specimen of each thickness was tested with point loads. Load test results indicated that the shear capacity associated with uniform load distribution was slightly less than the shear capacity for point loading of the 48-inch specimen.

The observation from Reference 6.4 and other literature that a uniform load distribution results in higher shear capacity may not apply for larger member depths. Reference 6.5 identified that uniform loading of 24-inch and 48-inch specimens was lower than corresponding tests performed with point loading. Considering these results, MPR concludes that uniform loading cannot be used to recover shear margin for the typical wall thicknesses in structures at Seabrook Station.

5.3 Reinforcement Anchorage Testing

The objectives of the Reinforcement Anchorage Test Program were to determine the effect of ASR on (1) the reinforcement anchorage performance (including lap splice), and (2) the flexural stiffness of reinforced concrete elements.

5.3.1 Test Description

The effects of ASR were evaluated using four-point bending tests to apply flexural load on large reinforced concrete beams that contained reinforcement splices at the longitudinal center of each beam (i.e., the constant moment region). The length of the reinforcement overlap (i.e., the lap splice) is specified by provisions in the ACI code, and was reflected in the test specimen design.

[redacted] test specimens were fabricated for this test program. One of these specimens was a control that was tested approximately 30 days following fabrication (i.e., prior to the development of ASR). The other [redacted] test specimens were allowed to develop ASR and were evaluated relative to the performance of the control test. Figure 5-6 shows the test setup for the reinforcement anchorage test specimens.

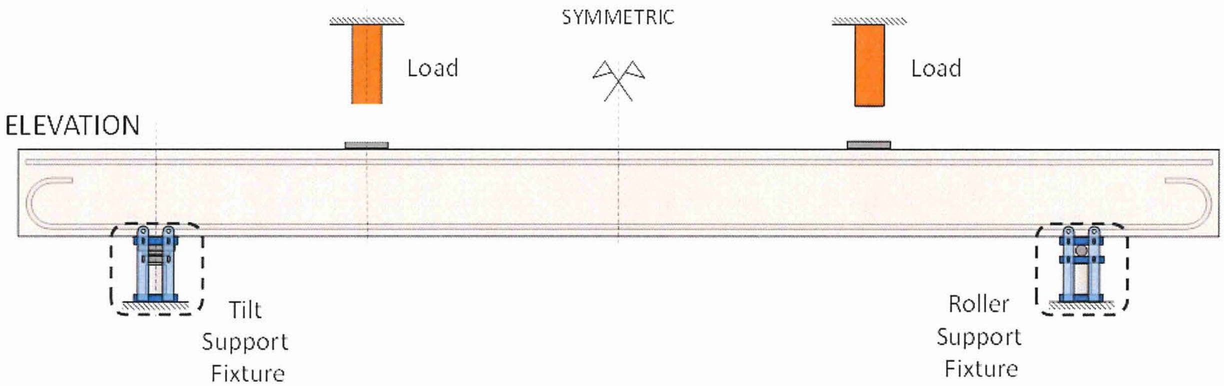

Figure 5-6. Test Setup for Reinforcement Anchorage Test Specimens (Elevation View)

Ideally, a concrete element with spliced reinforcing bars should perform similarly to elements with continuous reinforcement. Performance of the splice in the test specimens was considered satisfactory if the following criteria were met:

- Flexural yielding of the test specimens occurred at (or above) the theoretical "yield moment" (M_y), which is calculated by a moment-curvature analysis. Reinforced concrete members are designed such that the reinforcement will yield prior to failure. If the load applied to the test specimen results in a "yield moment" that is at least M_y, then the reinforcement has been developed up to its yield strength and the splice is performing like a continuous segment of reinforcement bar.

- Failure of the specimen occurs at or above its nominal flexural capacity (M_n), which is calculated using the provisions of ACI 318-71, and represents the maximum capacity of a flexural element. If the applied load to the test specimen demonstrates a flexural capacity of at least M_n, then the bond between the reinforcement bars and the concrete has not been adversely affected.

In summary, if both criteria are satisfied, then the presence of ASR has not adversely affected reinforcement anchorage or flexural capacity of the test specimen.

5.3.2 Test Results

Figure 5-7 provides load-displacement plots for the control test ([redacted]) and a test specimen that exhibited the highest level of expansion ([redacted]), which is typical of all ASR-affected specimens (total of [redacted] ASR-affected specimens).

[redacted]

Figure 5-7. Load-deflection Plots for Selected Reinforced Anchorage Test Specimens

The test results shown in Figure 5-7 indicate that ASR in the test specimens did not result in any adverse effect on the reinforcement anchorage capacity, although there is a change in the stiffness behavior, as shown by the lower deflection at flexural yielding and the absence of a notable slope change at low loads (~[redacted] kip) when flexural cracking begins.

Detailed evaluation identified that the criteria for satisfactory reinforcement anchorage performance were satisfied for each of the nine reinforcement anchorage tests. Specifically, the applied load resulted in a "yield moment" that exceeded the theoretical value (M_y) by [redacted]%, and the flexural capacity exceeded the nominal flexural capacity (M_n) by [redacted]%. The large number of tests and the repeatability of the data provide strong confidence in the conclusion that there was no adverse effect on reinforcement anchorage at the expansion levels tested.

5.3.3 Comparison to Literature

The published study discussed in Section 2.3.2 (Reference 1.9) included test results for reinforcement anchorage both with and without transverse reinforcement. Testing on specimens with transverse reinforcement indicated no significant loss of reinforcement anchorage strength, while testing on specimens without transverse reinforcement exhibited 40% decrease. Based on the results from the Reinforcement Anchorage Test Program, the test specimens actually behaved more like concrete with transverse reinforcement. Because the MPR/FSEL test program used a more realistic test method (e.g., flexural test of a large-scale beam containing a rebar splice, as compared to a rebar pullout test of a small specimen), specimens were more representative of structures at Seabrook Station, and the test results were highly repeatable, structural evaluations for Seabrook Station can use the MPR/FSEL conclusion (i.e., no loss of reinforcement anchorage) in lieu of the results from published literature.

5.3.4 Evaluation of Flexural Stiffness

The flexural behavior of a reinforced concrete element is non-linear over the full range of loading for two reasons: (1) changes in the stress-strain relationship of concrete in the tension zone as cracks initiate and grow and, (2) a non-linear (approximately parabolic) stress-strain relationship in the concrete compression zone. This behavior is illustrated in Figure 5-8, which shows a portion of the load-deflection response for the control test specimen.

[redacted]

Figure 5-8. Initial Part of Load Deflection Plot for Reinforcement Anchorage Control Specimen

Evaluation of the effect of ASR on flexural stiffness requires consideration of test specimen stiffness over the entire range of loading. Figure 5-8 identifies the following loads of interest:

- P_{crack} (Point B) is the load at which tensile stresses at the bottom of the test specimen (tension side) reach the tensile strength of concrete, resulting in flexural cracking.
- $P_{service}$ (Point D) is the load on the test specimen at the service-level condition (defined by ACI as 60 percent of the flexural yielding load).
- P_y (Point E) is the load corresponding to the flexural yielding of the test specimen.

The flexural stiffness of each test specimen over various regions can be calculated by finding the slope of the load-deflection plot between two selected points of reference.

Initial Flexural Stiffness

The initial flexural stiffness (prior to the onset of flexural cracking) is the slope from Point A to Point C (from Figure 5-8). This value provides a direct comparison to the calculated flexural stiffness, which is typically used in structural evaluations, and is referred to as the un-cracked concrete stiffness. Figure 5-9 shows the initial flexural stiffness for each test specimen relative to the theoretical value determined from material properties of the 28-day cylinders.

[redacted]

Figure 5-9. Effect of ASR-Related Expansion on Initial Flexural Stiffness

While Figure 5-9 shows a decrease in initial normalized flexural stiffness in the ASR-affected test specimens with respect to the control test specimen, there is no clear trend of changing stiffness as a function of through-thickness expansion. The decrease in initial stiffness may be due to the presence of small ASR-induced cracks at the onset of testing.

Service Level Flexural Stiffness

The service level flexural stiffness is the slope from Point A to Point D (from Figure 5-8), and represents the stiffness of the test specimen linearized from initial loading to the service level load (defined as 60 percent of the flexural yield load in ACI 318-71). This value is commonly used in reinforced concrete structural evaluations and is referred to as the cracked concrete stiffness. Modern design codes (ACI 318-11) allow the flexural stiffness of cracked beams and walls due to service loads to be taken as 0.35 times the nominal stiffness (EI). Figure 5-10 plots the measured flexural stiffness (normalized to the calculated flexural stiffness) as a function of through-thickness expansion.

[redacted]

Figure 5-10. Effect of ASR-Related Expansion on Service Level Flexural Stiffness

Figure 5-10 shows that the stiffness in ASR-affected test specimens is clearly greater than the control test specimen and that there is an increasing trend with respect to through-thickness expansion.

Summary of Results on Flexural Stiffness

The Reinforcement Anchorage Test Program provided data to assess changes in the flexural stiffness of reinforced concrete caused by development of ASR. Test results indicated that the initial flexural stiffness (i.e., prior to onset of flexural cracking) was generally lower than the theoretical value when ASR was present. However, the service level flexural stiffness, which is commonly used in structural evaluations, is within the limits specified by modern design codes.

5.3.5 Additional Testing - Retrofit for Reinforcement Anchorage

The original scope of the Reinforcement Anchorage Test Program included testing of retrofit concepts on specimens exhibiting ASR-induced expansion above which a deleterious effect was observed. A reduction in reinforcement anchorage was not observed at the expansion levels exhibited by the test specimens, so retrofit testing was not performed as part of the test program.

However, MPR and FSEL performed proof-of-concept testing on trial specimens (Reference 6.2). Specimens were fabricated with inadequate lap splice development length (relative to the ACI 318-71 requirement) to enable testing of a retrofit to augment reinforcement anchorage. The test specimens were comparable to those used in the Reinforcement Anchorage Test Program. The retrofit consisted of post-installed undercut anchors placed in the through-thickness direction that would behave like cast-in-place transverse reinforcement, confining the lap splice region. Retrofits were only installed from one side of the test specimen to simulate an actual structure where only one surface was accessible (e.g., underground structures at Seabrook Station).

Proof-of-concept testing was performed on four test specimens, as summarized in Table 5-2.

Table 5-2. Proof-of-Concept Testing for Reinforcement Anchorage Retrofit

Specimen	Lap Splice Development Length	Retrofit	Moment Capacity Relative to Design
AR0	Meets ACI 318-71 Requirement	No	1.13
AR1	Half of ACI 318-71 Requirement	No	0.83
AR2	Half of ACI 318-71 Requirement	Yes	0.98
AR3	Half of ACI 318-71 Requirement	Yes	1.02

The results indicated that the retrofit concept can increase the strength of a member with a deficient lap splice. However, specimens with the retrofit did not exhibit ductility that was comparable to the control specimen (AR0).

5.4 INSTRUMENTATION TESTING

The purpose of the Instrumentation Test Program was to evaluate the performance of several candidate instruments for measuring through-thickness expansion of reinforced concrete structures that have been affected by ASR.

5.4.1 Test Description

The Instrumentation Test Program evaluated three candidate instruments including one vibrating wire deformation meter (VWDM) and two extensometers. All instruments are installed in the concrete after core drilling to create a core bore.

- The VWDM consists of a vibrating wire strain gauge in series with a spring, which extends the effective range of the strain gauge. Measurements from the VWDM are performed using a battery-powered readout device. The observed expansion is calculated by comparing the readout device output with a baseline value recorded at the time of instrument installation.

- The snap ring borehole extensometer (SRBE) uses a spring-loaded, expanding snap ring to affix two anchors in a bore hole. A gauge rod of known length is connected to the base anchor (i.e., the deep anchor) and extends to the collar anchor (i.e., the shallow anchor). Expansion of the concrete is determined by using a calibrated depth micrometer to measure the distance between the reference surface on the collar anchor and the end of the gauge rod.

- The hydraulic borehole extensometer (HBE) uses a copper bladder, which is expanded with hydraulic fluid that is injected with a hand pump, to affix two anchors in the bore hole. A check valve in the fluid injection line maintains pressure in the bladder. Similar to the SRBE, a gauge rod of known length is connected to the base anchor and extends to the collar anchor. Expansion of concrete is determined by using a calibrated depth micrometer to measure the distance between the reference surface on the collar anchor and the end of the gauge rod.

The two types of extensometers were installed with [redacted] different gauge lengths, resulting in a total of [redacted] different configurations. Reduced length extensometers were investigated because they would not be installed as deep and would therefore reduce the risk of cutting rebar on the exterior reinforcement mat during installation.

To provide a point of reference to compare the expansion measured by each instrument, FSEL drilled companion holes through the entire thickness of the instrumentation specimen, such that each instrument location had companion holes on the left and right. A milled flat plate was placed on the opposite face of the beam to serve as a contact point for measurements with a depth gauge.

FSEL cast the instrumentation specimen in July 2014 and installed instruments on selected dates from August 2014 through May 2015. The test program concluded in July 2015. Staggering instrument installation investigated the impact of installing instruments after the onset of ASR (as will be the case at Seabrook Station).

5.4.2 Results

Based on the experience during the test program regarding quality of data, ease of installation, and reliability, the SRBE was identified as the best instrument for measuring through-thickness expansion at Seabrook Station.

Data Quality

Measurements obtained from the standard-length SRBE showed the best agreement with the reference measurements from the depth gauge. Instrument data agreed to within about [redacted]% with the reference measurements at expansion values below [redacted]%, which exceeds the range of estimated expansion levels currently observed at Seabrook Station (less than [redacted]%, based on information available at the time this report was published). Figure 5-11 presents the data obtained from the [redacted] standard-length SRBEs installed in the instrumentation specimen. The purple line represents SRBE measurements and the blue lines are the reference measurements (one dashed line for each companion hole; the solid line is the average). Other instruments exhibited irregular data that did not agree as well with the reference measurements (HBE, reduced length SRBE) or failed at higher levels of expansion (VWDM).

[redacted]

Figure 5-11. Comparison of SRBE Instrument Measurements with Depth Gauge Measurements

Figure 5-11 shows a large increase at the end of the test program for two of the four SRBEs. Those instruments were located nearer to the end of the beam where the wide cracking (as discussed in Section 4.2.3 and 4.2.5) occurred due to the lack of stirrups.

Ease of Installation

The SRBE and HBE were much easier to install than the VWDM, which requires refilling the volume around the instrument with grout after installation. Figure 5-12 illustrates the configuration of an installed SRBE.

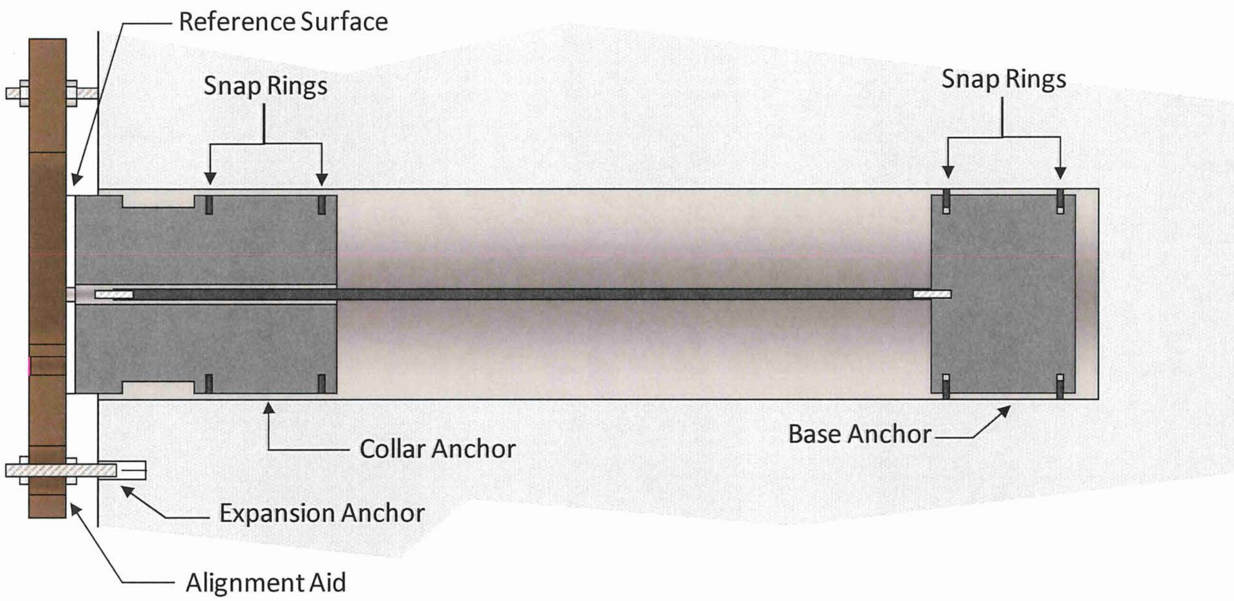

Figure 5-12. Illustration of SRBE during Installation

Long-Term Reliability

None of the SRBEs exhibited reliability problems during the test period. [redacted] of the [redacted] VWDMs stopped functioning after [redacted] Additionally, the VWDM is calibrated by the vendor but cannot be recalibrated following installation. FSEL observed slippage of the anchors for the HBEs, which resulted in erroneous measurements.

5.4.3 Conclusion

For the reasons listed above, MPR recommended normal-length SRBEs as the instrument for monitoring through-thickness expansion at Seabrook Station.

6 Implications for Seabrook Station

Results from the MPR/FSEL test programs will be used to support evaluations of ASR-affected reinforced concrete structures and future monitoring activities. This section summarizes the key implications for Seabrook Station identified as part of the MPR/FSEL test programs and related activities.

6.1 EXPANSION

6.1.1 Expansion Behavior

The reinforcement configuration of the test specimens in the large-scale test program included two-dimensional reinforcement mats in the in-plane directions to match most concrete structures at Seabrook Station. Expansion monitoring during the test programs identified that expansion will initially occur in all directions. However, after expansion in the in-plane directions reached [redacted]% to [redacted]%, the confinement provided by the reinforcement mats caused in-plane expansion to plateau. Subsequent expansion occurred primarily in the unreinforced through-thickness direction.

Technical literature (References 1.2, 1.3, & 1.13) and the MPR/FSEL test programs identified that expansion below [redacted]% [redacted] mm/m) does not result in significant structural consequences. Accordingly, expansion monitoring at Seabrook Station in only the in-plane directions is sufficient until expansion reaches [redacted]%, at which point through-thickness monitoring should begin.

The Structures Monitoring Program for Seabrook Station requires periodic visual inspections of all concrete surfaces. These inspections will identify new locations with ASR symptoms or existing locations with changing ASR symptoms. (Reference 2.5)

6.1.2 In-Plane Expansion Measurements

NextEra has been monitoring expansion of ASR-affected concrete at Seabrook Station using crack width measurement (i.e., combined cracking index (CCI)) since 2011. In the MPR/FSEL test programs, in-plane expansion monitoring of specimens included both CCI and measurement of the distance between pins embedded in the specimen during fabrication. The expansion values measured using embedded pins are a better measure of true engineering strain because these measurements reflect both material elongation and crack width. However, the test data showed that CCI and embedded pin measurements were in close agreement both in trend and magnitude, as the crack width measurements rapidly dominate the overall expansion. Therefore, use of CCI at Seabrook Station is a reasonable approximation for in-plane expansion since the beginning of plant life.

CCI is a labor-intensive methodology that may be cumbersome to maintain. As an alternative, NextEra could install embedded pins, which can be measured more rapidly with calipers, but will only provide expansion data from the time the pins are installed by taking the difference between the original distance between the pins and the measured distance. Adding this difference to the CCI measured at the time the pins are installed will provide an approximation for total in-plane expansion since the beginning of plant life.

6.1.3 Through-Thickness Expansion Measurements

The Instrumentation Test Program identified that the snap ring borehole extensometer (SRBE) is a reliable instrument that can provide accurate measurements of through-thickness expansion at Seabrook Station. The SRBE uses spring-loaded, expanding snap rings to affix two anchors in a bore hole. A gauge rod of known length is connected to the base anchor (i.e., the deep anchor) and extends to the collar anchor (i.e., the shallow anchor). Expansion of the concrete is determined by using a depth micrometer to measure the distance between the reference surface on the collar anchor and the end of the gauge rod.

6.1.4 Determining Total Through-Thickness Expansion

Installation of extensometers provides a means for monitoring expansion from the time that the instrument is installed. For structural evaluations at Seabrook Station, NextEra must be able to determine the total expansion from original construction.

In the MPR/FSEL test programs, material property testing of cylinders and cores representing the test specimens at various levels of ASR development identified that modulus of elasticity is a sensitive and repeatable indicator of through-thickness expansion. MPR-4153 (Reference 2.6) provides a methodology for using this observation to enable Seabrook Station to determine total through-thickness expansion, as follows:

- When the extensometer is installed, determine the elastic modulus of the concrete by material property testing of cores removed from the structure at the extensometer location.
- Establish the original elastic modulus by either (1) using the ACI 318-71 correlation to calculate elastic modulus from the 28-day compressive strength records, or (2) obtaining cores from representative ASR-free locations and testing for elastic modulus.
- Calculate the reduction in elastic modulus by taking the ratio of the current elastic modulus of the ASR-affected area to the original elastic modulus.
- Determine through-thickness expansion from original construction to the time the extensometer is installed using an empirical correlation. The correlation relates reduction in elastic modulus with measured expansion from test specimens used during the large-scale ASR structural testing programs. The recommended method in MPR-4153 applies a reduction factor of [redacted] to the elastic modulus ratio, which results in a conservatively high calculation of pre-instrument expansion.

- Calculate current total expansion by adding the current extensometer measurements to the expansion at the time of instrument installation.

6.1.5 Recommendations for Implementation

Execution of a multi-year large-scale test program to support evaluation of ASR-affected reinforced concrete structures is unique in the nuclear industry in purpose, scale, and methodology. Application of the results of the MPR/FSEL test programs requires that the test specimens be representative of reinforced concrete at Seabrook Station, and that expansion behavior of concrete at the plant be similar to that observed in the test specimens. Test specimen design addressed representativeness of the test specimens, and promoted expansion behavior consistent with the plant (e.g., use of two-dimensional reinforcement mats). MPR recommends that NextEra perform checks to ensure that expansion behavior at Seabrook Station is similar to expansion behavior of the FSEL test specimens, as follows:

- Inspect cores obtained for determining through-thickness expansion for mid-plane cracks. As discussed in Section 4.2.3, the test specimens did not exhibit large cracking between the reinforcement mats away from the specimen edges.

- Perform routine inspections of in-plane expansion, through-thickness expansion, and volumetric expansion and compare results to the limits of the test program. Application of the test results beyond the limits of the test program would require further evaluation.

- Periodically compare expansion behavior trends at Seabrook Station with observations to FSEL test specimens. Appendix B of this report provides guidelines for the approach and content of these periodic comparisons. MPR recommends that an initial comparison be performed in the near term after extensometers are installed. MPR recommends follow-up comparisons at least 5 years prior to the Period of Extended Operations (PEO) and every 10 years thereafter[9].

- At least five years prior to PEO and 10 years thereafter, remove cores for 20% of the extensometer locations and compare through-thickness expansion determined from the modulus-expansion correlation determined from the extensometer and the original modulus result. Appendix C of this report provides guidelines for the approach and content of these corroboration studies.

6.2 Structural Performance

This section summarizes the conclusions of the test programs that can be used for structural evaluations. A companion report (MPR-4288, "Seabrook Station: Impact of Alkali-Silica Reaction on the Structural Design Basis") describes the effect of ASR on the structural design basis of affected structures at Seabrook Station and provides guidance for evaluations of those structures.

[9] As an example, the PEO will begin in 2030. If the next assessment is performed 5 years prior to PEO in 2025, subsequent assessments would be performed in 2035 and 2045.

6.2.1 Anchors and Embedments

Results from the Anchor Test Program indicate that there is no reduction of anchor capacity in ASR-affected concrete with in-plane expansion levels of less than [redacted] mm/m. The current maximum in-plane expansion observed at Seabrook Station is considerably less than this expansion level. Because the two-dimensional reinforcement mats at Seabrook Station should cause in-plane expansion to plateau at relatively low levels, it is unlikely that ASR will cause expansion of [redacted] mm/m.

In-plane expansion due to ASR creates microcracks parallel to the axis of an anchor, which are most pronounced in the concrete cover. These microcracks that open perpendicular to the concrete surface have the potential to provide a preferential failure path within a potential breakout cone, leading to degraded anchor performance. Conversely, through-thickness expansion has the potential to create microcracks perpendicular to the axis of an anchor. These potential microcracks that open parallel to the concrete surface do not provide a preferential failure path to result in degraded anchor performance. Test results confirmed that anchor performance was insensitive to through-thickness expansion of up to about [redacted]%. Accordingly, MPR recommends in-plane expansion (e.g., via CCI) as the monitored parameter for assessing anchor performance.

6.2.2 Shear Performance

Results from the Shear Test Program indicate that there is no reduction of shear capacity in ASR-affected concrete with through-thickness expansion levels up to [redacted]% or volumetric expansion levels up to [redacted]%, which are the maximum expansion levels exhibited by the test specimens. The [redacted] ASR-affected test specimens (total of [redacted] tests) were all capable of reaching their calculated shear strength per ACI 318-71. The test results indicated a repeatable trend that higher levels of ASR resulted in higher shear capacity due to ASR-induced prestress. For conservatism, MPR does not recommend taking credit for this prestressing as part of structural evaluations.

6.2.3 Reinforcement Anchorage

Results from the Reinforcement Anchorage Test Program indicate that there is no reduction in the performance of reinforcement lap splices in ASR-affected concrete with through-thickness expansion levels up to [redacted]% or volumetric expansion levels up to [redacted]%, which are the maximum expansion levels exhibited by the test specimens. The [redacted] ASR-affected test specimens were all capable of reaching their calculated flexural strength per ACI 318-71, and the yield and bending moments were relatively insensitive to the level of ASR-induced expansion.

6.2.4 Flexural Stiffness

While progression of ASR in the reinforcement anchorage test specimens did not impact the yield or ultimate flexural capacity of the test specimens, there was a notable change in the stiffness, characterized by a decrease in deflection at yield. Key observations on the changes in flexural stiffness included the following:

- The service level flexural stiffness is the value commonly used in reinforced concrete structural evaluations and is referred to as the cracked concrete stiffness. Modern design codes (ACI 318-11) allow the flexural stiffness of cracked beams and walls due to service loads to be taken as 0.35 times the nominal stiffness (EI). The test program results indicated that all ASR-affected test specimens exceeded this stiffness value.

- The flexural stiffness of the ASR-affected specimens was less than that of the control test specimen at loads less than [redacted]% of the load at which the test specimen yielded. The reduction is attributed to the presence of numerous ASR-induced cracks in the test specimen prior to the application of the load during the structural tests.

- The flexural stiffness between the onset of flexural cracking and flexural yielding was observed to be greater in the ASR-affected test specimens compared with the control test specimen and showed a generally increasing trend with the increase in ASR-related expansion at the time of structural test. The increased stiffness with the progression of ASR is attributable to the ASR-induced prestressing in the test specimens.

The impact on seismic performance resulting from these differences in flexural stiffness will be evaluated as part of the companion report (MPR-4288).

6.2.5 Use of Structural Test Program Results

Applicability to Site Structures

Results of the MPR/FSEL test program are generally applicable to all reinforced concrete structures at Seabrook Station, which have similar reinforcement configurations and concrete mixture designs. This approach was corroborated by material property testing of the various test specimens for the MPR/FSEL test programs, which had minor differences in reinforcement ratio and number of stirrups on specimen ends, and were fabricated from different concrete batches (although the mix designs were comparable). Observed material properties exhibited a consistent relationship between aging and expansion across the various beam designs, which suggests that the aging mechanism is insensitive to the specific boundary conditions of a particular specimen design. This conclusion supports application of structural performance results from the MPR/FSEL test programs to the range of structures at Seabrook Station.

Interpretation of Threshold Expansion Values

The MPR/FSEL test program results provide threshold expansion values for which ASR has no effect on the respective limit state. These values reflect the extent of ASR development that was achieved as part of the test programs; they do not represent limits above which ASR has a deleterious effect. Expansion at Seabrook Station is currently well below these threshold expansion values. If expansion approaches the threshold expansion values, NextEra may perform additional research to justify structural adequacy beyond the ASR development levels evaluated in the MPR/FSEL test programs.

6.2.6 Retrofit Testing

Proof-of-concept testing for potential retrofits provided insights that would have supported subsequent qualification testing of retrofits on ASR-affected test specimens for shear and reinforcement anchorage. However, because the test specimens did not exhibit any degradation in structural performance, the retrofits were not tested on ASR-affected specimens.

If ASR-related expansion at Seabrook Station approaches the maximum expansion identified in the test programs and additional actions are necessary to justify structural adequacy, NextEra may pursue follow-up testing of the retrofits to demonstrate their efficacy in ASR-affected concrete.

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4. Test Reports for MPR/FSEL Test Programs

4.1. MPR-3722, "Strength Testing of Anchors in Concrete Affected by Alkali-Silica Reaction," Revision 2. (Seabrook FP# 100718)

4.2. MPR-4262, "Shear and Reinforcement Anchorage Testing of Concrete Affected by Alkali-Silica Reaction," Volume I, Revision 1 & Volume II, Revision 0. (Seabrook FP# 100994)

4.3. MPR-4231, "Instrumentation for Measuring Expansion in Concrete Affected by Alkali-Silica Reaction," Revision 0. (Seabrook FP# 100972)

5. Commercial Grade Dedication Report for MPR/FSEL Test Programs

5.1. MPR-3726, "Commercial Grade Dedication Report for Seabrook ASR Anchor Testing," Revision 0. (Seabrook FP# 100719)

5.2. MPR-4247, "Commercial Grade Dedication Report for Seabrook ASR Anchor Testing (Block Series and Girder Series Phase 2)," Revision 0. (Seabrook FP# 100986)

5.3. MPR-4259, "Commercial Grade Dedication Report for Seabrook ASR Shear, Reinforcement Anchorage, and Instrumentation Testing," Revision 0. (Seabrook FP# 100995)

5.4. MPR-4286, "Supplemental Commercial Grade Dedication Report for Seabrook ASR Test Programs," Revision 0. (Seabrook FP# 101003)

6. Documentation for Information Only Testing at FSEL

6.1. Letter from FSEL (Bayrak) to MPR (Simons), "Morgan Therese Allford's Research," dated April 21, 2016.

6.2. Beiter, K., "Retrofit of Deficient Lap Splice with Post-Installed Anchors," University of Texas at Austin, December 2015.

6.3. Dondrea, A., "Undercut and Grouted Anchors as Post-Installed Shear Reinforcement," University of Texas at Austin, August 2014.

6.4. Dassow, N., "Effect of Uniform Load on the Shear Strength of Slender Beams without Shear Reinforcement," University of Texas at Austin, August 2014.

6.5. Klein, J., "Behavior of Slender Beams without Stirrups: Effects of Load Distribution and Member Depth," University of Texas at Austin, December 2015.

A
Test Specimens

This appendix provides photographs, diagrams, and drawings for the test specimens used in the Anchor, Shear, Reinforcement Anchorage, and Instrumentation Test Programs. (References 4.1, 4.2, & 4.3)

Figure A-1. Photo of Girder Series Anchor Test Specimen

[redacted]

Figure A-2. Photo of Block Series Anchor Test Specimen with Anchors Installed

[redacted]

Figure A-3. Diagram of Block Series Anchor Test Specimen Showing Reinforcement

[redacted]

Figure A-4. Diagram of [redacted]-Inch Shear Test Specimen Showing Reinforcement

[redacted]

Figure A-5. Diagram of Reinforcement Anchorage Test Specimen Showing Reinforcement

[redacted]

Figure A-6. Diagram of Instrumentation Test Specimen Showing Reinforcement (Elevation View)

[redacted]

Figure A-7. Diagram of Instrumentation Test Specimen Showing Reinforcement (Plan View)

B

Guidelines for Periodic Expansion Behavior Check

1. PURPOSE

This appendix provides guidelines for performing periodic checks of observed expansion behavior at Seabrook Station to confirm that expansion behavior is consistent with FSEL test specimens.

2. BACKGROUND

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

To confirm that expansion behavior at Seabrook Station is similar to the FSEL test specimens, MPR recommends (in Section 6.1.5) that NextEra perform periodic checks of expansion behavior at Seabrook Station and compare observations from the MPR/FSEL test programs.

MPR recommends that an initial check be performed after extensometers are installed, and follow-up checks were recommended at least 5 years prior to the Period of Extended Operations (PEO) and every 10 years thereafter

3. CHECK 1 - REVIEW OF CORES FOR MID-PLANE CRACKING

As ASR developed in the FSEL test specimens, a large crack was noted in the center of the surfaces of the beam that were between the reinforcement mats. The FSEL test specimens did not exhibit large cracking between the reinforcement mats away from the specimen edges. In all cases, the large crack penetrated only a few inches into the specimen height. The observed cracking was therefore attributed to an edge effect.

The large surface crack is not representative of expansion behavior of the large majority of structures at Seabrook Station, which have a network of members that are either cast together or integrally cast with special joint reinforcing details. The confinement provided by the network of members in a structure is likely sufficient to preclude large cracks like those seen in the FSEL test specimens. However, the surface crack could be observed in the top surface of a wall if there are no stirrups spanning across the tops of the reinforcement mats. In such cases, the crack will extend only a few inches from the top surface.

As recommended in Section 6.1.5, NextEra should inspect cores for mid-plane cracks upon removal of cores. As part of the periodic check of expansion behavior, NextEra should review documentation of all cores obtained more recent than the last periodic check for any trends in observation of mid-plane cracks. The objective of the inspection is to confirm the absence of mid-plane cracking away from a surface in the through-thickness direction. Observation of a mid-plane cracks initiated by a mechanism other than the edge effect would be unexpected and would prompt an evaluation to determine appropriate follow-up actions.

4. Check 2 - Expansion Relative to Test Program Limits

The MPR/FSEL test programs included structural testing of reinforced concrete specimens with a range of ASR development. The conclusions of the test program are applicable to reinforced concrete at Seabrook Station that is within the range of ASR development tested at FSEL.

4.1. Summary of Test Program Limits

The limits of ASR development evaluated by the MPR/FSEL testing and are provided in Table B-1.

Table B-1. Summary of Test Program Limits

Parameter	Limit	Basis
In-Plane Expansion	[redacted] mm/m ([redacted]%)	Anchor Test Program
Through-Thickness Expansion	[redacted]%	More Conservative of the Shear and Reinforcement Anchorage Test Programs
Volumetric Expansion	[redacted]%	More Conservative of the Shear and Reinforcement Anchorage Test Programs

4.2. Margin for Future Expansion

Routine monitoring of ASR-affected locations will identify if the observed expansion at Seabrook Station exceeds the limits in Table B-1, and would necessitate a location-specific structural evaluation. As part of the periodic check, MPR recommends that NextEra determine the potential for future expansion to exceed the limits. This review of margin to the MPR/FSEL test program limits may be performed by considering the "expansion rate" observed over a series of measurements and the projected time to reach the test program limits.

NextEra's review should include consideration of the uncertainty associated with extensometer readings and with in-plane expansion measurements. Assessments of "expansion rate" for the purpose of projecting future expansion should rely on trends comprised of multiple data points. If such projections indicate that the limits may be exceeded prior to the next periodic check, NextEra should further investigate the location(s) in question or develop contingency plans for extending the expansion limit (e.g., supplemental testing).

4.3. Calculation of Volumetric Expansion

Volumetric strain is determined by adding the observed strain in each of the three directions (Reference 1.14), as follows:

$$\varepsilon_v = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$$

Where:

ε_v = volumetric strain
ε_1 = principal strain (e.g., in the length direction)
ε_2 = principal strain (e.g., in the height direction)
ε_3 = principal strain (e.g., in the depth direction)

For the parameters monitored at Seabrook Station, this equation can be re-written, as follows:

$$\varepsilon_v = 2 \times (0.1 \times CCI) + \varepsilon_{TT}$$

Where:

ε_v = volumetric strain, %
CCI = combined cracking index, mm/m
ε_{TT} = through-thickness expansion, %

Using this expression for the FSEL test specimens, the maximum volumetric expansion of a shear test specimen was [redacted]% and the maximum volumetric expansion of a reinforcement anchorage test specimen was [redacted]%. The more conservative of the two, [redacted]%, was selected as the volumetric expansion limit. Figure B-1 illustrates the volumetric expansion limit.

[redacted]

Figure B-1. Volumetric Expansion Limit

Note that the in-plane expansion limit of [redacted] mm/m is bounded by the volumetric expansion limit in Figure B-1. If all of the [redacted]% volumetric expansion were in the in-plane direction, the CCI would only be [redacted] mm/m.

5. CHECK 3 - EXPANSION DIRECTION

For the FSEL test specimens, the rate of expansion was approximately the same in all three directions until expansion reached [redacted]% to [redacted]% (i.e., [redacted] to [redacted] mm/m). Thereafter, the FSEL test specimens exhibited much greater expansion in the through-thickness direction than the in-plane directions. These observations led to a conclusion that in-plane monitoring by crack width summation (i.e., CCI) sufficiently characterizes ASR development until at least [redacted]% expansion (i.e., [redacted] mm/m), after which through-thickness monitoring is required to track further ASR expansion. NextEra has installed extensometers in selected locations where in-plane expansion is less than 1 mm/m.

For locations where NextEra has installed an extensometer, NextEra should check the trend for expansion direction as a confirmation of consistency with the expansion behavior observed in the MPR/FSEL test program.

NextEra has installed several extensometers in locations where in-plane expansion is less than 1 mm/m. This provides the opportunity to check consistency of expansion behavior over the entire range exhibited at Seabrook Station.

Figure B-2 is a chart that may be used for analyzing the trend for observed expansion direction at Seabrook Station.

[redacted]

Figure B-2. Expansion Direction Trend Chart

Plotting of expansion data at Seabrook Station onto a chart like Figure B-2 is expected to result in a "cloud" of data that exhibits considerable variability. For the FSEL test specimens, the point at which expansion reoriented primarily in the through-thickness direction varied between specimens, which were essentially identical. Data from Seabrook Station may exhibit further variability from differences in configuration (e.g., wall thickness) and confinement (e.g., from deadweight).

NextEra should perform an engineering evaluation if the periodic expansion check identifies either of the following circumstances:

- Any location with CCI less than [redacted] mm/m exhibits through-thickness expansion approaching the test program limit (i.e., greater than [redacted]%). Such an observation would challenge the premise that an extensometer is not needed for locations with a CCI of less than [redacted] mm/m. The engineering evaluation would focus on the suitability of this criterion.

- The general trend of expansion behavior at Seabrook Station significantly departs from the expansion behavior of the FSEL test specimens. The expected trend at Seabrook Station is that in-plane and through-thickness expansion values will be comparable at lower expansion levels and eventually transition to predominately through-thickness expansion.

Other factors may cause the apparent in-plane expansion at Seabrook Station to exceed the observed in-plane expansion of the FSEL test specimens (Reference 2.7) and should be considered in the engineering evaluation. Measurement of in-plane expansion for some locations at Seabrook Station is not directly comparable to that from the MPR/FSEL test programs. At Seabrook Station, external loads (e.g., load applied by expansion from backfill), drying shrinkage, and thermal expansion and contraction can initiate cracking or exacerbate (i.e., open up) existing cracking, both of which impact in-plane expansion measurements. In contrast, the MPR/FSEL test programs isolated the effect of ASR, so the in-plane cracking was predominantly from expansion of ASR gel. All expansion measurements from the MPR/FSEL test programs were prior to the application of an external load. Structural calculations can be used to help identify applicable non-ASR factors that may influence in-plane expansion at a given location.

MPR recommends that NextEra also review petrography results to determine if the petrographer noted details that were relevant to expansion behavior. Petrography results that alter NextEra's understanding of expansion or concrete degradation at a given location (e.g., impact of non-ASR factors) should be considered as part of the expansion assessment and should be referenced for use in future engineering evaluations.

C
Guidelines for Corroboration Study

1. Purpose

This appendix provides a guideline for the in-plant corroboration of the methodology for determining through-thickness expansion of ASR-affected structures. In support of this objective, this appendix also reviews the approach for developing the correlation using data from the MPR/FSEL test programs and the methodology for using the correlation that was recommended in MPR-4153 (Reference 5).

2. Through-Thickness Expansion Monitoring at Seabrook Station

NextEra has installed extensometers in selected monitoring locations throughout Seabrook Station. The extensometers allow NextEra to monitor through-thickness expansion that occurs from the time that the instrument is installed through the end of plant life.

To calculate the cumulative through-thickness expansion since original construction, the extensometer measurement must be added to the expansion up to the time the instrument is installed (i.e. pre-instrument expansion). Pre-instrument expansion is determined using a correlation between reduction in elastic modulus and ASR-induced expansion that was presented in MPR-4153 (Reference 2.6).

MPR-4153 defined the correlation based on a regression analysis that gives a best fit of the data from the MPR/FSEL test programs. MPR compared the correlation to literature data from various sources (References 1.16, 1.17, 1.18, 1.19, 1.20, 1.21, and 1.22). The literature data compare favorably with the Seabrook-specific correlation, and therefore validate application of the correlation at the plant (Reference 2.6).

To provide appropriate conservatism, the methodology described in MPR-4153 prescribes reducing the normalized elastic modulus by [redacted]%. This adjustment drives the calculated pre-instrument expansion higher, which is in the direction of conservatism. This adjustment is used for assessing concrete relative to the through-thickness expansion acceptance criterion. Figure C-1 shows the correlation and the conservative effect of applying the [redacted]% adjustment to the normalized elastic modulus.

[redacted]

Figure C-1. Correlation between Elastic Modulus and Through-Thickness Expansion

3. Process for Determining Through-Thickness Expansion

For each extensometer location, cores are taken to obtain corresponding data for modulus of elasticity at the time the extensometer was installed. These data are used to calculate pre-instrument expansion at each location using the best-fit correlation (ε_0) and with the adjustment to the normalized elastic modulus (ε_{0_adj}). Figures C-2 and C-3 provide examples illustrating how these values are obtained for a hypothetical data point where the elastic modulus at the time of extensometer installation was [redacted] of the original elastic modulus value (i.e., normalized elastic modulus, E_n, is [redacted]).

[redacted]

Figure C-2. Determination of Best-Estimate Pre-Instrument Through-Thickness Expansion

[redacted]

Figure C-3. Determination of Adjusted Pre-Instrument Through-Thickness Expansion

4. METHODOLOGY FOR IN-PLANT CORROBORATION STUDY

To supplement the comparison of the correlation to literature data that was documented in MPR-4153 (Reference 2.6), NextEra should conduct an in-plant corroboration study.

In the future, additional cores will be taken in the vicinity of selected extensometers for elastic modulus testing. For each location selected, MPR recommends that two specimens be tested and the results averaged to determine the best-estimate elastic modulus at the time of the corroboration study[10]. These test results will be used to determine the change in through-thickness expansion since installation of the extensometers and compare it to the change determined from extensometer readings.

This section describes the detailed procedure for performing the corroboration study and includes an example with graphical illustrations of how the results will be interpreted. The corroboration study will analyze the data in two different ways (i.e., Test 1 and Test 2) to enable assessment of

[10] In accordance with the methodology in MPR-4153, companion compressive strength testing is performed.

the data obtained at the time of the corroboration study and also the data obtained at the time the extensometer was installed.

4.1. Test 1 – Assessment of Data Obtained at Time of Study

The approach for Test 1 assumes that the through-thickness expansion determined at the time of extensometer installation is correct and evaluates the data point obtained at the time of the corroboration study.

The elastic modulus test results will be used to determine the normalized elastic modulus for a particular location at the time of the corroboration study, and the best-estimate total through-thickness expansion using the best-fit correlation (ε_{t_EM}). Figure C-4 provides an example for a normalized elastic modulus of [redacted] at the time of the corroboration study.

[redacted]

Figure C-4. Determination of Best-Estimate Through-Thickness Expansion Using Elastic Modulus for Corroboration Study

Through-thickness expansion will also be determined using the extensometer, in accordance with the methodology for routine monitoring (Table 1-1). Specifically, the differential expansion ($\Delta\varepsilon_{inst}$) measured using the extensometer at the time of the corroboration study will be added to the adjusted through-thickness expansion at the time the extensometer was installed

($\varepsilon_{0\_adj} + \Delta\varepsilon_{inst} = \varepsilon_{t\_inst}$). (For routine monitoring, the pre-instrument expansion is based on the adjusted correlation from MPR-4153 to provide conservatism.)

Figure C-5 provides an example illustrating the method for calculating ε_{t_inst} using the hypothetical data point of E_n = [redacted] when the extensometer was installed and assuming a measured differential expansion of [redacted]%.

[redacted]

Figure C-5. Determination of Through-Thickness Expansion Using Extensometer for Corroboration Study

The through-thickness expansion determined using the extensometer (ε_{t_inst}) will be compared to the best-estimate expansion using the correlation from MPR-4153 (ε_{t_EM}). The result of Test 1 is satisfactory if $\varepsilon_{t\_EM} \leq \varepsilon_{t\_inst}$. This result indicates that the expansion monitoring methodology is providing an appropriate level of conservatism.

Figure C-6 provides a graphical illustration of how the results are compared for Test 1.

[redacted]

Figure C-6. Example Application of Acceptance Criterion for Test 1

4.2. Test 2 – Assessment of Data from Extensometer Installation

Test 2 assumes that the through-thickness expansion determined at the time of the corroboration study is correct, and evaluates the data point obtained at the time of extensometer installation. The approach for Test 2 is essentially the reverse of Test 1.

Test 2 uses the same data from elastic modulus testing as was used for Test 1. Different from Test 1, the elastic modulus is used to determine the adjusted total expansion at the time of the corroboration study using the adjusted correlation (ε_{t_adj}). Figure C-7 provides an example for a normalized elastic modulus of [redacted] at the time of the corroboration study.

[redacted]

Figure C-7. Determination of Adjusted Through-Thickness Expansion Using Elastic Modulus for Corroboration Study

Like Test 1, the differential through-thickness expansion at the time of the corroboration study will be determined using the extensometer ($\Delta\varepsilon_{inst}$), in accordance with the methodology for routine monitoring from the ASR AMP. However, for Test 2, this value will be subtracted from the adjusted through-thickness expansion determined at the time of the corroboration study ($\varepsilon_{t\_EM\_adj}$ - $\Delta\varepsilon_{inst} = \varepsilon_{0\_inst}$).

Figure C-8 provides an example illustrating the method for calculating ε_{0_inst} using the hypothetical data point of E_n = [redacted] when the corroboration study is performed and assuming a measured differential expansion of [redacted]%.

[redacted]

Figure C-8. Determination of Initial Through-Thickness Expansion Using Extensometer and Elastic Modulus Data from Corroboration Study

The calculated initial through-thickness expansion (ε_{0_inst}) will be compared to the best-estimate through-thickness expansion at the time of extensometer installation (ε_0, illustrated in Figure C-1), as shown in Figure C-8. The result of Test 2 is satisfactory if $\varepsilon_0 \leq \varepsilon_{0\_inst}$. This result indicates that the expansion monitoring methodology is providing an appropriate level of conservatism.

Figure C-9 provides a graphical illustration of how the results are compared for Test 2.

[redacted]

Figure C-9. Example Application of Acceptance Criterion for Test 1

4.3. Acceptable Range of Elastic Modulus Values

The corroboration study checks that the correlation from MPR-4153 is an appropriate representation of expansion behavior at Seabrook Station. Corroboration would be unsuccessful if either of the following two conditions exist:

- Through-thickness expansion determined by the correlation is **much greater** than through-thickness expansion determined using the extensometer. Test 1 confirms that this condition does not exist.

- Through-thickness expansion determined by the correlation is **much less** than through-thickness expansion determined using the extensometer. Test 2 confirms that this condition does not exist.

Example Showing Acceptable Range of Normalized Elastic Modulus

Using both tests establishes a range of acceptable elastic modulus values for the cores obtained for the corroboration study. For the example provided above, where the normalized elastic modulus at the time of initial extensometer placement is [redacted] and the measured expansion from the extensometer is [redacted]%, the acceptable bounds would be as follows:

- For Test 1, the acceptance criterion would be met if the best-estimate expansion using the correlation at the time of the corroboration study is less than [redacted]%. This result corresponds to a normalized elastic modulus of no less than [redacted] for the core taken at the time of the corroboration study. Figure C-10 illustrates a result that would satisfy this criterion with no margin.

- For Test 2, the acceptance criterion would be met if the initial expansion, calculated by subtracting the differential expansion measured by the extensometer from the adjusted expansion determined using the correlation, is greater than [redacted]%. This result corresponds to a normalized elastic modulus of no greater than [redacted] for the core taken at the time of the corroboration study. Figure C-11 illustrates a result that would satisfy this criterion with no margin.

[redacted]

Figure C-10. Example Showing Minimum Acceptable Normalized Elastic Modulus

[redacted]

Figure C-11. Example Showing Maximum Acceptable Normalized Elastic Modulus

Enclosure 8 to SBK-L-18072

NextEra Energy Seabrook, Application for Withholding Proprietary Information from Public Disclosure and Affidavit

NextEra Energy Seabrook, LLC

AFFIDAVIT IN SUPPORT OF APPLICATION FOR WITHHOLDING PROPRIETARY INFORMATION FROM PUBLIC DISCLOSURE

County of Rockingham)
)
State of New Hampshire)

I, Eric McCartney, being duly sworn according to law, depose and state the following:

(1) I am the Regional Vice President – Northern Region of NextEra Energy Seabrook, LLC (NextEra Energy Seabrook), and have been delegated the function of reviewing the information described in paragraph (3) which is sought to be withheld, and have been authorized to apply for its withholding.

(2) I am making this Affidavit in conjunction with NextEra Energy Seabrook's "Application for Withholding Proprietary Information from Public Disclosure" accompanying this Affidavit and in conformance with the provisions of 10 CFR Section 2.390.

(3) The information sought to be withheld is contained in Enclosures 6 and 7 of NextEra Energy Seabrook's letter SBK-L-18072, Eric McCartney (NextEra Energy Seabrook) to U.S. Nuclear Regulatory Commission, entitled "Revised Structures Monitoring Aging Management Program."

(4) The information sought to be withheld is considered to be proprietary and confidential commercial information because alkali-silica reaction (ASR) is a newly-identified phenomenon at domestic nuclear plants. The information requested to be withheld is the result of several years of intensive NextEra Energy Seabrook effort and the expenditure of a considerable sum of money. This information may be marketable in the event nuclear facilities or other regulated facilities identify the presence of ASR. In order for potential customers to duplicate this information, similar technical programs would have to be performed and a significant manpower effort, having the requisite talent and experience, would have to be expended. The extent to which this information is available to potential customers diminishes NextEra Energy Seabrook's ability to sell products and services involving the use of the information. Thus, public disclosure of the information sought to be withheld is likely to cause substantial harm to NextEra Energy Seabrook's competitive

position and NextEra Energy Seabrook has a rational basis for considering this information to be confidential commercial information.

(5) The information sought to be withheld is being submitted to the NRC in confidence.

(6) The information sought to be withheld has, to the best of my knowledge and belief, consistently been held in confidence by NextEra Energy Seabrook, has not been disclosed publicly, and not been made available in public sources.

(7) The information is of a sort customarily held in confidence by NextEra Energy Seabrook, and is in fact so held.

(8) All disclosures to third parties, including any required transmittals to the NRC, have been or will be pursuant to regulatory provisions and/or confidentiality agreements that provide for maintaining the information in confidence.

I declare that the foregoing affidavit and the matters stated therein are true and correct to the best of my knowledge, information, and belief. Further, the affiant sayeth not.

Sincerely,

Eric McCartney
Regional Vice President – Northern Region
NextEra Energy Seabrook, LLC
626 Lafayette Road
Seabrook, New Hampshire 03874

Subscribed and sworn to before me
this 18 day of May, 2018.

Notary Public
Notary Public
My commission expires Jan. 14, 2020

SHIRLEY SWEENEY
MY COMMISSION EXPIRES JAN. 14, 2020
NOTARY PUBLIC
NEW HAMPSHIRE