Enclosure 2 to SBK-L-12106

The Evaluation, "Impact of ASR on Concrete Structures and Attachments" (Foreign Print 100716)

MPR
ASSOCIATES INC.
ENGINEERS

MPR-3727
Revision 1
(Seabrook FP # 100716)
May 2012

Seabrook Station: Impact of Alkali-Silica Reaction on Concrete Structures and Attachments

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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RECORD OF REVISIONS

Revision	Affected Pages	Description
0	All	Initial Issue
1	All	Editorial corrections/clarifications, reissue of the calculation in Appendix A, and incorporation of additional Figures (3-1 & 6-2) to aid the reader.

Executive Summary

NextEra Energy has identified the presence of pattern cracking typical of Alkali-Silica Reaction (ASR) in multiple Seismic Category I structures at Seabrook Station. ASR can be explained simply as the reaction between silica from the aggregate and alkali constituents in the cement or the pore solution. This reaction produces a gel that expands as it absorbs moisture. Expansion of the gel exerts tensile stress on the concrete resulting in cracking. ASR cracking may degrade the mechanical properties of the concrete necessitating an assessment of the adequacy of the structures and supports anchored to the structures.

This report evaluates the near-term adequacy of concrete structures affected by ASR and system/component anchorages in ASR-affected concrete at Seabrook Station. The evaluation addresses:

- the effect of ASR on the structural demand and seismic response of the concrete buildings,
- the potential for local failure of individual concrete components (e.g., walls or slabs), and
- the effect of ASR on the capacity of the concrete anchors and embedments.

Confinement provided by reinforcing steel and other restraints (e.g. deadweight of the structure) is a key factor in assessing the impact of ASR on reinforced concrete structures. Confinement limits ASR expansion of the *in situ* structure, which reduces the extent of deleterious cracking and the resultant reduction in concrete mechanical properties. Given this interplay between expansive ASR degradation and structural restraint, the structural assessment herein relies on structural testing rather than typical materials type testing on concrete cores removed from the structure.

The conclusion of our assessment is that, given the current extent of ASR cracking, the reinforced structures at Seabrook Station remain suitable for continued service for at least an interim period. This conclusion is based on the following considerations.

- ASR has a negligible effect on the structural demand and seismic response of the reinforced concrete structures at Seabrook Station.
- Although there may have been some reduction in structural capacity, the reduction is less than that necessary to challenge the suitability of the structures for operation during an interim period.
  - Results from a comprehensive walkdown effort show that the extent of ASR cracking in the great majority of areas in the plant is sufficiently low and that published guidance indicates that detailed evaluations are not necessary in such cases.

- For the areas that had sufficient cracking to merit a detailed evaluation, the great majority either have positive margin or sufficient margin that can likely be recovered to accommodate potential effects of ASR degradation.
- Given the conservatism in the evaluation methodology and the fact that the available test data on effects of ASR on reinforced concrete components are for small-scale tests that are not representative of a large structure, there is reasonable assurance that structures are suitable for continued service.

- There is little reduction in anchor capacity at the maximum cracking levels observed in the plant. Any small reduction in capacity is readily offset by conservatisms in the design capacity of the anchors, or by crediting the average 28-day compressive strength for concrete at Seabrook Station instead of the specified strength.

The assessment herein will be further supported through (1) full-scale testing programs regarding shear and lap splice performance for elements without transverse reinforcement; and (2) a comprehensive anchor testing program.

Finally, this report identifies follow-on actions that will provide an assessment of the long-term adequacy of plant structures and anchorages and provide for Aging Management Program (AMP) parameters for extended plant operations.

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1 Introduction

1.1 Purpose

This report evaluates the near-term adequacy of concrete structures affected by Alkali-Silica Reaction (ASR) degradation, and attachments (i.e., anchorages) in ASR-affected concrete at Seabrook Station. The evaluation addresses:

- the effect of ASR on the structural demand and seismic response of the concrete buildings,
- the potential for local failure of individual concrete components (e.g., walls or slabs), and
- the effect of ASR on the capacity of the concrete anchors and embedments.

The evaluation herein focuses on the near-term adequacy of plant structures and system/component anchorages. In addition, this report identifies follow-on actions necessary to assess the long-term adequacy of plant structures and anchorages and define Aging Management Program (AMP) parameters. These follow-on actions may include: monitoring, remediation, structural testing programs and additional analyses.

1.2 Background

1.2.1 Overview of Plant Structures

Site Overview

The structures at Seabrook Station are laid out in a highly vertical arrangement. Many structures have underground areas that are at least 40 feet below grade. Several structures have underground areas that are 80 feet below grade. It is MPR's understanding that the site preparation required blasting into the granite. A first cut was made to bowl out the granite to satisfy the required depth for the majority of the structures. Deeper shafts were required for several structures, extending from the first cut down to the required depth for the remaining structures. After the structures were constructed, the gaps between the structural walls and the granite were backfilled with lean concrete, which essentially "locked" the structures into the bedrock.

Given the depth of the excavations relative to the water table, a dewatering system was necessary during plant construction. This system was abandoned once construction was complete as building design features were intended to prevent ingress of groundwater into the buildings.

A waterproofing membrane surrounds most structures. The membrane was applied to the exterior surface of the walls below grade prior to backfilling with lean concrete. This membrane serves as a barrier against water ingress. However, early in plant life, some areas began to show signs of groundwater ingress, which suggests that portions of the membrane are not performing as designed or may have been compromised.

Building Design

The majority of the structures at Seabrook Station are reinforced concrete structures. These structures were designed and constructed to comply with the 1971 edition of ACI 318, *Building Code Requirements for Reinforced Concrete* (Reference 9.1.1). The subset of structures that house the reactor system, safety systems and other equipment and facilities necessary to achieve and maintain shutdown of the plant (i.e. Seismic Category I) are designed to withstand the loadings from external events such as the design basis seismic event. For buildings that are very close together, there are small gaps (~3 inches) between structures that are filled with a flexible material to seismically isolate the adjacent structures.

1.2.2 Alkali-Silica Reaction Concern at Seabrook

Overview of Alkali-Silica Reaction

ASR is the reaction between silica from the aggregate and alkali constituents in the cement. The reaction produces a gel that expands as it absorbs moisture. Expansion of the gel exerts tensile stress on the concrete resulting in cracking. Typical cracking resulting from ASR is described as "pattern" or "map" cracking and is usually accompanied by a dark staining adjacent to the cracks at the surface of the structure.

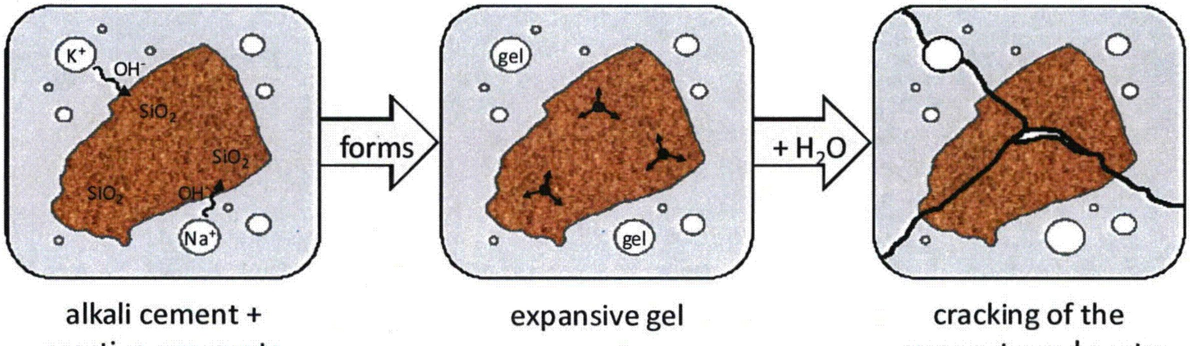


Figure 1-1. ASR Mechanism (Reference 9.5.1)

The cracking may degrade the mechanical properties of the concrete necessitating an assessment of the adequacy of the structures and supports anchored to the structures. As noted in References 9.5.4 and 9.5.5, the concrete properties most rapidly affected are the elastic modulus and the tensile strength.

Reinforcing steel, loads on the concrete structure (e.g., deadweight of the structure) and the configuration of the structure can restrain expansion of the gel and thereby limit the resultant concrete cracking. Given that the impact of ASR on mechanical properties relates to the cracking, constraint of the expansion in effect limits the reduction of mechanical properties *in situ*. As will be discussed in Section 4, a concrete core removed from a wall will show a greater degradation of mechanical properties compared to the *in situ* structure because the core loses its structural context (i.e. confinement) once it is removed from the wall.

ASR at Seabrook Station

NextEra Energy personnel initially identified pattern cracking typical of ASR in the B Electrical Tunnel in 2009, and subsequently, several other Seismic Category I structures. As a result, NextEra Energy implemented multiple campaigns to remove concrete cores from the walls in several plant structures to confirm the presence of ASR. Petrographic examination of the cores identified the telltale signs of ASR. Further, mechanical property testing of the unconfined cores showed an apparent decrease in mechanical properties of the concrete, particularly the elastic modulus, which is consistent with ASR degradation.

The concrete used at Seabrook was not expected to be susceptible to ASR due to the following: (1) the coarse aggregate is igneous rock that passed the ASR reactivity testing used during construction; (2) low-alkali cement (<0.6% total alkali) was used; and (3) the aggregate passed petrographic examination per ASTM C295. The American Society for Testing and Materials (ASTM) standard test procedures ASTM C289 and C227 were used to assess aggregate reactivity during construction. These ASTM standards were the appropriate tests at the time of construction, but it is now known that these tests may not accurately predict the reactivity of slow-reacting aggregates. A combination of aggregate being more susceptible to ASR than originally thought and groundwater intrusion during plant life appears to have resulted in the observed ASR.

1.3 Strategy for Addressing ASR

The long-term adequacy of concrete structures at Seabrook Station that have been affected by ASR is being addressed using a combination of elements as listed below. This approach is consistent with published guidance for managing ASR degradation of structures (see References 9.5.4, 9.5.5 and 9.5.6) and accounts for the importance of confinement of ASR cracking provided by steel reinforcement (see Reference 9.5.1).

- Characterize the extent of ASR degradation at Seabrook Station through the combination of:
  - engineering walkdowns of plant structures,
  - petrographic examination of cores removed from plant structures, and
  - testing of cores removed from plant structures.
- Assess the impact of ASR on plant structures using test data regarding the structural performance of ASR-affected concrete components.
- Implement test programs to supplement the current body of knowledge regarding the impact of ASR on the performance of reinforced concrete structures and on the capacity of anchors embedded in reinforced concrete affected by ASR.

- Monitor the progression of ASR through Seabrook Station's Structural Monitoring Program, as well as an Aging Management Program specific to ASR.

- Remediate areas with significant degradation as required to ensure plant structures and equipment anchorages remain adequate to accommodate all design basis loading conditions.

- Investigate means to address water ingress to reduce and potentially arrest future degradation from ASR, to the extent practicable given the plant's layout and hydrology.

The strategy for demonstrating the long-term adequacy of concrete structures involves two evaluations. The initial evaluation assesses concrete structures at Seabrook Station to determine if continued operation can be justified for an interim period, until the full-scale structural testing programs are complete and any required monitoring programs are implemented. The second evaluation will assess the long-term adequacy of the concrete structures considering the results of the full-scale structural testing program, other in-progress test programs and results from periodic monitoring of the structures.

1.4 Scope of Report

This report focuses exclusively on the structural implications of ASR, assessing the adequacy of the plant structures for an interim period until data from the structural testing programs are available. The data from the structural testing programs will support assessment of the long-term adequacy of ASR-affected structures at Seabrook Station, and development of key inputs for the Structural Monitoring Program and the ASR Aging Management Program.

This report discusses the walkdowns and anchor test program to the extent necessary to support the structural assessment. The detailed results of the walkdowns are documented in Reference 9.2.9. The program for testing anchors in ASR-affected concrete is documented in Reference 9.2.6.

This report does not cover evaluations of measures to address water ingress as these efforts are being handled separately by NextEra Energy.

2 Summary of Results and Conclusions

Confinement provided by reinforcing steel and other restraints is a key factor regarding the impact of ASR on reinforced concrete structures. Confinement limits ASR expansion of the *in situ* structure, which reduces the extent of deleterious cracking and the resultant reduction in concrete properties. Given this interplay between expansive ASR degradation and structural restraint, the structural assessment herein relies on structural testing rather than typical materials type testing on cores removed from the structure.

2.1 Impact of ASR on Structures

This assessment of the impact of ASR on structures considers both the impact on structural demand and the impact on structural capacity. The assessment uses structural test data available in published literature. These data are for small-scale test specimens with configurations that are not necessarily representative of Seabrook Station, but provide conservative results. Notwithstanding these differences, the test results are applied as they are the best data available at this time. The test programs underway will perform full-scale tests of configurations that are representative of the walls in the B Electrical Tunnel. A more definitive evaluation of the impact of ASR on the structures will be made once these test programs are complete.

2.1.1 Structural Demand

While ASR generally reduces the stiffness of unreinforced concrete, reinforced concrete structures affected by ASR behave differently from unreinforced cores due to confinement of the concrete by the reinforcing steel. In fact, ASR has been shown to increase the stiffness of reinforced concrete sections, at least for structures with confining reinforcement in all three directions (Reference 9.5.1, Section 4.3). Further, these same tests show no difference in the response in the linear-elastic range. Overall, we conclude that the impact of ASR on structural demand and seismic response of the reinforced concrete structures at Seabrook Station is negligible. Some of the bases for this conclusion are noted below.

- The natural frequency of structures is proportional to the square root of the structure's stiffness, thereby reducing the percent change in natural frequency for a given change in material stiffness.

- Changes in stiffness associated with ASR only affect dynamic loads such as seismic. While seismic loads are significant in some areas, the fact that ASR does not affect the other loads that contribute to the total loads reduces the overall impact of ASR on governing load combinations. While changes in elasticity can affect load distribution in redundant structures (e.g., monolithic concrete), the structural components at Seabrook Station were originally analyzed using relatively simple load assumptions and load distributions. Therefore, a change in elasticity will not significantly affect the load distribution in structures.

Since the ASR has a negligible impact on structural demand, the impact of ASR on structures and on structural attachments can be assessed solely on the basis of changes in capacity.

2.1.2 Structural Capacity

Although ASR pattern cracking can be observed in many areas within Seismic Category I structures and Maintenance Rule structures, only a limited portion of these areas have sufficient ASR degradation to merit detailed evaluation. The MPR walkdown effort encompassed 131 locations[1] with the potential for pattern cracking. Of these 131 locations, only 24 exceeded the screening criterion of a Combined Cracking Index of 1.0 mm/m. Based on published guidance for addressing ASR, concrete with a Combined Cracking Index below 1.0 mm/m is generally considered to have only minimal impact on structural capacity. Of the 24 areas above the 1.0 mm/m Combined Cracking Index, 11 were selected for detailed evaluation. The sample was biased to include those with the highest Combined Cracking Indices (i.e., >1.5 mm/m) and those in the scope of present operability determinations.

Detailed evaluations of these 11 areas focused on out-of-plane shear and reinforcement lap splices (anchorage of reinforcing steel), two limit states for which available data indicated that there is a potential decrease in capacity due to ASR.

- Out-of-Plane Shear—Available data from scale tests indicate that ASR can potentially reduce shear capacity by up to 25%. However, ACI 318-71, on average, includes approximately 50% margin on the shear capacity for components up to two feet thick, but lesser margin for components thicker than 2 feet.
  - For components up to two feet thick, ASR should not degrade shear capacity below that calculated from ACI 318-71 as the margin inherent in the code exceeds the maximum reduction in shear capacity.
  - For components greater than two feet thick, ASR may degrade shear capacity up to 25% below that calculation from ACI 318-71.
- Lap Splices and Embedment—Available data from rebar pullout tests, an outdated and unreliable test method, indicate a 40% strength reduction for lap splices in ASR-affected concrete. However, there is approximately 23% conservatism in the ACI 318-71 equations for lap splice strength. Therefore, ASR could decrease lap splice strength about 17% relative to that calculated using ACI 318-71. (For cases where the actual concrete compressive strength was credited, the 23% conservatism in the ACI 318-71 equations for lap strength cannot be credited as part of this conservatism derives from the difference between specified and actual compressive strength.)

[1] Note that the walkdown scope did not include all candidate locations within Seismic Category I structures and the selected Maintenance Rule structures. Some of the candidate locations were eliminated from the scope for detailed walkdown on the basis of a general site walkdown identified no indications of ASR or significant water ingress.

For the 11 areas subjected to detailed evaluations, a total of 143 evaluations were assessed to determine if there was sufficient margin to accommodate ASR. Of these 143 evaluations, 47 (33%) do not have sufficient margin based on the margin documented in the Seabrook Station calculations. However, after exploring means for potentially recovering margin, only 15 of the 143 evaluations (10%) appear to have insufficient margin to accommodate ASR.

Given the conservative nature of our approach, the fact that 15 evaluations appear to have insufficient margin to accommodate ASR degradation does not necessarily mean that the respective structures are not suitable for continued service. The conservative aspects of our evaluation include the following:

- The 40% reduction of lap splice strength in ASR-affected concrete is not representative of the expected lap splice performance in ASR-affected concrete at Seabrook Station.

  - The 40% reduction is based on a test method which is outdated and known to be unrealistic. In Reference 9.1.2, ACI Committee 408 indicates that the rebar pullout test is "the least realistic" test of the four test methods that they evaluated. Further, they state that:

    > *"...the use of pullout test results as the sole basis for determining development length is inappropriate and not recommended by Committee 408."*

    The 40% reduction value was applied despite this admonishment as it is conservatively derived from the most relevant data available.

  - The test used reinforcing steel much smaller than typical reinforcing steel used at Seabrook Station. Reinforcement anchorage is known as a limit state that does not scale well.

  - Although the level of ASR degradation in the tests was not documented, the test program targeted advanced levels of ASR degradation. The ASR at Seabrook Station is not at an advanced state.

  The conservatisms in the evaluation approach, coupled with the ACI 408 Committee's strong statement on the suitability and reliability of rebar pullout testing suggest that there is significant uncertainty in the screening criterion that was applied. It is concluded that there is reasonable assurance that the structures are adequate for an interim period.

- Potential strength reductions of 25% for out-of-plane shear are not 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%. The average impact is a reduction of 6%, which is within the available margin for all areas.

  - The shear capacity reduction due to ASR of 25% is based on a small-scale test using 5-inch x 3-inch beams. It is well known that shear phenomenon does not scale well.

Therefore, the reduction in shear capacity due to ASR is likely less than the 25% used in the screening.

It is noted that test programs have been initiated to determine the shear capacity and lap splice performance in full-scale, ASR-affected specimens that replicate the key features of the walls in the B Electrical Tunnel. The tests are essentially proof tests that will provide a definitive assessment of the nominal margin inherent in the design and any apparent reduction due to ASR.

NextEra Energy performed a supplemental assessment for the 15 evaluations that initially appeared to have insufficient margin to accommodate ASR. The objective of their assessment was to demonstrate adequate margin. Their evaluation focused on conservatism in the demand (i.e. loads and load factors).

2.2 Impact of ASR on Anchors

Assessment of the impact of ASR on anchors is based on testing that MPR sponsored at the Ferguson Structural Engineering Laboratory (FSEL) at the University of Texas at Austin. The objective of the testing was to better understand the performance of post-installed anchors (both expansion and undercut) under tension when subjected to a range of ASR-induced cracking. Both the pullout/pull-through and concrete breakout failure mechanisms were investigated. The expansion anchors tested were from the Hilti Kwik Bolt family, which is a common type of anchor used at Seabrook Station. The undercut anchors tested were Drillco Maxi-Bolts, which are used in some applications at Seabrook. These undercut anchor results also provide insights for other types of anchors including embedments and cast in place anchors.

The initial testing, which was intended to provide results to support this interim assessment, used an ASR-affected bridge girder available at FSEL. Future test series will use new blocks in which ASR is grown over time; the blocks will provide a much more representative simulation of concrete walls at Seabrook Station.

The key conclusion from the tests is that there is little reduction in anchor capacity at the cracking levels observed in the plant. For expansion anchors there was no reduction in pullout/pull-through capacity at the Combined Cracking Indices observed at Seabrook Station. For undercut anchors there was up to a 16% reduction in capacity relative to the control tests. This potential reduction in capacity is readily offset by conservatisms in the design capacity of the anchors, or by crediting the average 28-day compressive strength for concrete at Seabrook Station instead of the specified strength. It is concluded that the range of ASR-induced cracking currently observed at Seabrook Station does not adversely impact the operability of safety-related concrete anchors in service at the plant.

2.3 Path Forward

The path forward for addressing the ASR issue at Seabrook Station is to complete the test programs that will provide information necessary to (1) support long-term assessment of the impact of ASR on plant structures, and (2) define action levels for the Structural Monitoring Program and the ASR Aging Management Program. These test programs include:

- A Shear Test Program to establish the shear capacity and flexural stiffness of concrete beams without transverse (i.e. shear) reinforcement, which have varying levels of ASR degradation. This test program will also investigate potential structural modification concepts to restore shear capacity if necessary.

- A Lap Splice Test Program to establish the performance of reinforcement anchorage and flexural stiffness in concrete beams without transverse reinforcement which have varying levels of ASR degradation. The testing will also investigate potential structural modification concepts to compensate for apparent degradation of reinforcement anchorage, if necessary.

- An Anchor Test Program to establish the performance of expansion anchors and undercut anchors in ASR-affected concrete. The next phase of the program will use concrete specimens that are representative of walls at Seabrook Station.

3 Characterization of ASR Degradation

Since the first identification of ASR pattern cracking at Seabrook Station, NextEra Energy has implemented a series of efforts to characterize the extent of ASR degradation at the site. These efforts include:

- multiple campaigns to remove concrete cores from the areas exhibiting ASR pattern cracking for petrographic examination, and for testing to assess the effect on concrete mechanical properties; and
- engineering walkdowns of Seismic Category I structures and some 10CFR50.65 Maintenance Rule structures to assess the condition of concrete structures, focusing on evidence of ASR and evidence of moisture which could lead to expansion due to ASR.

In addition to the above efforts, NextEra Energy is initiating a test program on aggregate reactivity to assess whether the reaction is near exhaustion or will continue into the future.

3.1 Examination and Testing of Cores

NextEra Energy has implemented three campaigns to obtain cores from plant structures at Seabrook Station. The first campaign addressed the B Electrical Tunnel, which was the first place where ASR pattern cracking was observed. The second campaign expanded the scope to five additional areas: below-grade portions of the Containment Enclosure Building (CEB), the Emergency Feedwater Pumphouse, the Diesel Generator Building, the Residual Heat Removal (RHR) & Containment Spray (CS) Equipment Vault, and RCA Walkway. The third campaign obtained cores for testing to resolve differences in compressive strength testing from the previous results for the B Electrical Tunnel.

3.1.1 Petrographic Examination

B Electrical Tunnel

Simpson, Gumpertz, & Heger (SGH) performed petrographic examination of four concrete cores from the B Electrical Tunnel. SGH identified occasional cracks visible without magnification and numerous microcracks visible under low magnification in the coarse aggregate particles and the surrounding paste structure. The microcracks are often interconnected, forming a network of pattern cracking. The cracks were often filled with a white material that appeared to be ASR gel. In addition, SGH identified dark rims around the perimeters of the coarse aggregate particles (i.e., "reaction rims") that are consistent with ASR. Based on these observations, SGH concluded that the concrete distress was caused by ASR (Reference 9.2.1).

Other Locations
As part of an extent of condition evaluation, NextEra Energy obtained cores from other locations exhibiting symptoms of ASR in concrete similar to the B Electrical Tunnel. Specifically, NextEra Energy obtained cores from the Radiologically Controlled Area (RCA) Walkway, the RHR & CS Equipment Vault, the Emergency Feedwater Pumphouse, the Diesel Generator Building, and the Containment Enclosure Building. SGH analyzed these cores and determined that cores from all of the examined locations except the RCA Walkway exhibited evidence of ASR, including pattern cracking, internal aggregate fractures (some partially filled with ASR gel), and dark reaction rims around aggregate (Reference 9.2.5).

As discussed in Reference 9.3.3, SGH performed petrographic examinations on sections of three 16" partial-depth concrete cores from the interior face of a B Electrical Tunnel wall (24" thick). The objective of this testing was to determine if the degree of ASR varied through the thickness of the wall. There was a higher degree of ASR cracking in the samples of the concrete near the exposed interior wall surfaces (i.e., cover) as compared to concrete removed from deeper within the wall.

3.1.2 Mechanical Testing of Cores

NextEra Energy obtained several sets of cores for mechanical testing to investigate the presence of ASR. Results of these mechanical tests are summarized as follows:

- Miller Engineering & Testing performed compressive strength testing of twelve concrete cores from the B Electrical Tunnel. The as-tested compressive strength values ranged from 3,630 psi to 5,690 psi with an average of 4,790 psi. All reported values exceed the original minimum specified compressive strength of 3,000 psi, but were lower than the original construction compressive strength test results of 6,120 psi. (Reference 9.2.2)

- NextEra Energy contracted SGH to perform compressive strength testing on the remnants of four concrete cores from the previous round of tests. The as-tested compressive strength values ranged from 5,790 psi to 6,360 psi. (Reference 9.2.3)

- SGH also performed testing for elastic modulus. The as-tested elastic modulus values ranged from 1.95×10^6 psi to 2.25×10^6 psi, which are lower than the expected elastic modulus values. ACI 318-11 (Reference 9.1.3, Section 8.5.1) calculates the elastic modulus as a function of compressive strength; commentary to ACI 318-11 (Reference 9.1.3, Section R8.5.1) indicates the tolerance on the modulus is ±20%. For concrete with a specified compressive strength of 3,000 psi, the equation ($E_c = 57{,}000 \times \sqrt{f_c'}$) calculates an elastic modulus of 3.12×10^6 (range of 2.50×10^6 to 3.74×10^6 psi if the ±20% tolerance is applied). For the minimum measured compressive strength of 5,790 psi, the expected elastic modulus is 4.33×10^6 psi. (Reference 9.2.3)

- Based on the substantial differences between the measurements from Miller and SGH, NextEra Energy contracted Wiss, Janney, Elstner (WJE) to perform testing for compressive strength on twelve ASR-affected cores and three control cores (i.e., no evidence of ASR) from the B Electrical Tunnel. The as-tested compressive strength values ranged from 4,720 psi to 6,610 psi. (Reference 9.2.4)

- SGH performed testing for elastic modulus and compressive strength on cores from other locations at Seabrook as part of an extent of condition evaluation. Specifically, cores from the RCA Walkway, the RHR & CS Equipment Vault, the Emergency Feedwater Pumphouse, the Diesel Generator Building, and the Containment Enclosure Building were tested. Elastic moduli were lower than expected in all areas except the RCA Walkway due to ASR; however, compressive strength results were consistent with the original compressive strength for each location tested. (Reference 9.2.8)

Mechanical testing of cores was initially pursued by NextEra Energy, because this approach is a traditional method for determining mechanical properties of existing concrete structures per ACI 228.1R. However, the results of this testing are not indicative of the structural performance of an ASR-affected structure. As discussed in Reference 9.5.1, in most circumstances, the measured strength of a core will be less (if not significantly less) than the strength of the concrete in the structure. Cores are no longer subject to the strains imposed by ASR-related expansion or restraints imposed by the reinforcing cage, and therefore do not accurately represent the structural behavior. Accordingly, the reduction of mechanical properties observed in the mechanical tests are not representative of the structural performance of buildings at Seabrook Station.

The mechanical tests are useful as a diagnostic tool to confirm that ASR is present. As discussed in Reference 9.5.4, mechanical properties of concrete are negatively affected by ASR to varying extents. Typically, the elastic modulus of unconfined concrete is one of the most rapidly affected mechanical properties; compressive strength is affected less rapidly. The test results obtained by Seabrook are consistent with this expectation and support a diagnosis that ASR is present.

3.1.3 Residual Aggregate Reactivity Testing

NextEra Energy is initiating a test program to determine the residual aggregate reactivity. This program includes both mortar testing in accordance ASTM C 1260 and concrete prism testing in accordance with ASTM C 1293. Both tests monitor expansion over the test to determine whether a particular aggregate is suitable for new construction.

- ASTM C 1260, Potential Alkali Reactivity of Aggregate (Mortar-Bar Method) – This test uses a 1N sodium hydroxide solution and very high temperature (176°F) to rapidly react silica in the aggregate during a 16-day test.

- ASTM C 1293, Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction – This test specifies preparation of a concrete prism using the aggregate under investigation and a sodium hydroxide admixture to supply alkali reactant. The test specimen is stored in a warm, humid environment (i.e., in a sealed container over water at 38°C [100°F]) to accelerate the reaction. This test requires at least one year to obtain results, but is more reliable than the ASTM C 1260 testing, which uses more aggressive test conditions.

This testing is planned for reclaimed coarse aggregate from cores that had been used for compressive strength testing. The samples will use coarse aggregate reclaimed from cores that were affected by ASR and aggregate reclaimed from cores that were not affected by ASR. This testing will provide a qualitative assessment of the amount of reactive silica remaining (i.e., the relative extent of reaction) by comparing test results for aggregate in ASR-affected areas and results for aggregate in control areas that are unaffected by ASR (or fresh aggregate from the quarry). If reactive silica is the limiting reactant, the tests will provide a qualitative assessment of the potential for additional ASR expansion in the concrete structures as Seabrook Station. If, however, alkali is the limiting reactant in the Seabrook Station concrete, the potential for future expansion will be less than that estimated from the reactive aggregate tests.

3.2 FIELD WALKDOWNS

MPR completed a comprehensive site walkdown effort to assess the extent of ASR throughout the plant. Prior to the walkdowns, petrographic examination of concrete cores from the first two campaigns had confirmed the presence of ASR in five Seismic Category I structures. The primary objectives for the walkdowns were to:

- Identify and assess any apparent degradation from ASR, including estimating *in situ* expansion,
- Assess whether concrete in the vicinity of supports for safety-related Systems, Structures, or Components (SSCs) shows any indications of ASR distress, and
- Document and characterize water intrusion or evidence of previous water intrusion since this condition is a key contributor to concrete deterioration and distress caused by ASR.

The results of these walkdowns constitute a baseline condition assessment of plant structures which will be used for trending the progression of ASR. Note that the walkdowns are not a comprehensive structural inspection per ACI guidelines, as such an inspection is covered by Seabrook Station's Structural Monitoring Program.

3.2.1 Walkdown Scope

The overall scope for the walkdowns focuses on Seismic Category I structures as well as some 10CFR50.65 Maintenance Rule structures given their significance for nuclear safety. The areas of interest are primarily those areas that are potentially exposed to moisture either by groundwater ingress (exterior walls below grade, base slabs), high humidity in the area, or exposure to precipitation and ambient humidity (exterior walls above grade). Many plant areas are not exposed to moisture (interior walls, especially above grade) and have a very low risk of developing cracking due to ASR; these plant areas were excluded from the walkdown scope.

The walkdown scope was separated into three phases to represent locations that require increasing levels of effort for assessment. Phase 1 walkdowns included locations in Category I and Maintenance Rule structures that were readily accessible and susceptible to ASR. Locations requiring scaffolding or confined space permits were included in Phase 1. Phase 2 walkdowns included selected locations in Category I and Maintenance Rule structures where the concrete surface was accessed by removing the coating and cleaning the concrete surface (typically for a 3' by 3' area). The areas that concrete surfaces were accessed beyond the coating for further assessment in Phase 2 were selected by a preliminary walkdown of coated areas during Phase 1 walkdowns. The selection parameters utilized a biased screening, seeking out areas that showed evidence of coating distress or water accumulation behind the coating. Phase 3 walkdowns include locations in Category I and Maintenance Rule structures that are normally inaccessible for walkdowns (e.g., inside manholes, high radiation areas, etc). Phase 3 areas are likely to only be assessed in parallel with concurrent activities and coincidental outage related opportunities (e.g., removal of missile barrier at the CEB, opening of manholes, etc.). The full list of structures (and rooms within) identified for walkdown assessment may be found in *Scope for Alkali-Silica Reaction Walkdowns*, (Reference 9.7.1).

The Phase 1 and Phase 2 walkdowns were performed from August 2011 to February 2012, and are documented in Reference 9.2.9. Phase 3 walkdowns will be performed when the areas can be accessed.

3.2.2 Implementation

The walkdowns were performed in accordance with *Procedure for Alkali-Silica Reaction Walkdowns and Assessment Checklist* (Reference 9.7.2). The procedure focuses on identifying evidence of ASR, and evidence of moisture, either past or present, which could lead to deleterious expansion from ASR. It includes quantification of the extent of ASR cracking. The procedure is consistent with published guidance for the initial condition assessment of structures affected by ASR. Key elements of the procedure are described below.

Pattern Cracking

Concrete deleteriously affected by expansive ASR is characterized by a network or "pattern" of cracks (Reference 9.5.3). ASR involves the formation of an alkali-silica gel which expands when exposed to water. Microcracking 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 (Reference 9.5.4). 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. To identify and verify the presence of ASR the maximum crack width, a cracking index, and a description of the cracking including any visible surface discoloration were documented.

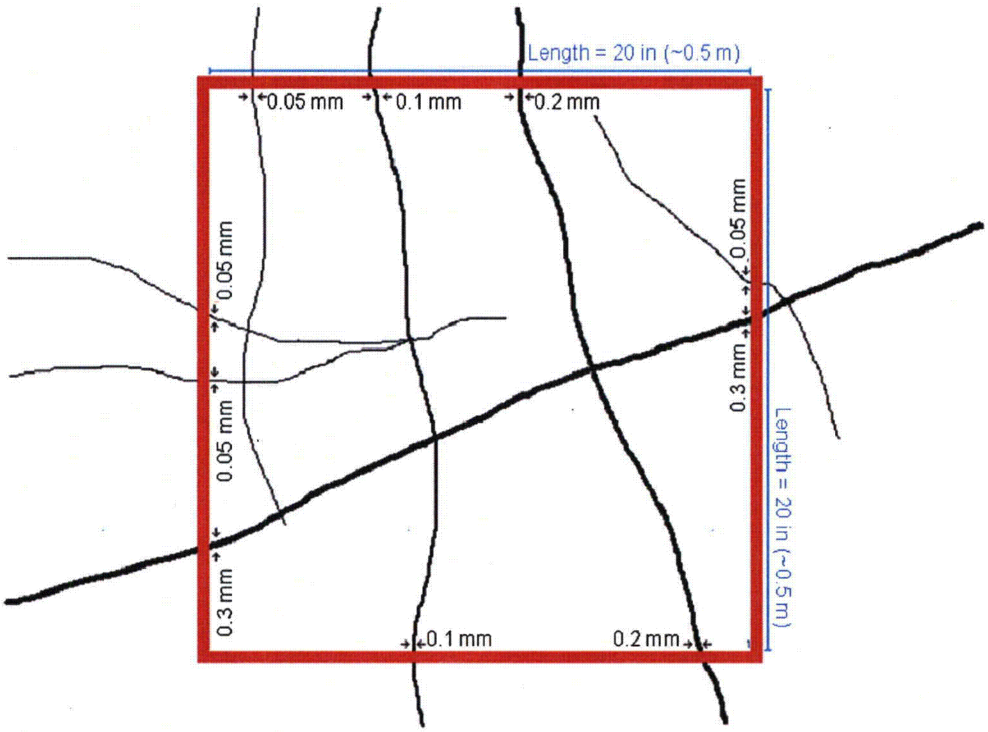
Additional Cracking

Any non-ASR cracks within five feet of an apparent degradation area were documented in case the structural assessment needs to consider the ASR concurrent with non-ASR degradation. The maximum crack width and a general description of the cracking were included.

Cracking Index

Cracking Indices were determined for accessible surfaces exhibiting ASR pattern cracking. The Cracking Index used in the walkdowns is consistent with the definition in *Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures*, (Reference 9.5.4). The Cracking Index is the summation of the crack widths on the horizontal or vertical sides of a 20-inch by 20-inch square on the ASR-affected concrete surface. Since each side of the square is 0.5 m, the Cracking Index in a given direction is reported in units of mm/m. A cracking index of 1 mm/m can be equivalent to 1 millistrain of expansion if straining between the cracks are ignored as an engineering approximation. This approximated strain value is not a precise measurement of strains experienced due to ASR expansion. The Cracking Index is most useful as an indicator of relative expansions experienced by various areas.

The horizontal and vertical Cracking Indices were determined as shown in Figure 3-1. However, for the data presented in this report, the horizontal and vertical Cracking Indices were averaged to obtain a Combined Cracking Index (CCI) for each area of interest. The CCI represents the expansion along the entire perimeter of the 20-inch by 20-inch square. Review of the walkdown results reveals little difference between the horizontal and vertical Cracking Indices at a given location with few exceptions. Therefore, use of the CCI does not alter the conclusions of the assessment documented herein.[2]



Combined Cracking Index (CCI) = Sum of Crack Widths (mm) / Sum of Side Lengths (m)
CCI = 1.40 mm / 2.0 m = 0.7 mm/m

Figure 3-1. Example of Combined Cracking Index (CCI) Measurements

Water Ingress

Published studies on ASR and discussions with recognized experts indicate that external sources of water (groundwater ingress, precipitation) are not always required to produce ASR. However,

[2] As discussed in Section 7, use of the CCI facilitates comparison to anchor bolt testing in ASR-affected concrete bridge girders. The girders have rebar in only the vertical direction so there is a significant difference between horizontal and vertical Cracking Indices.

petrographic examination of cores from internal walls show no evidence of ASR and general walkdowns of the plant show that an external source of water is necessary to produce ASR distress in the concrete used at Seabrook. As such, any areas with evidence of seepage (past or present) were documented and described (e.g. staining, discoloration, or efflorescence). Along with seepage, any evidence of areas of foreign material ingress, including suspected ASR gel that were within or near an apparent degradation area were noted.

Popouts
A popout is caused by a fragment breaking out of the surface of the concrete, leaving a hole varying in size that contains a fractured aggregate particle (Reference 9.5.3). Popouts caused by expansive ASR are formed as a result of the pressure induced by ASR gel. The number, size, and location of popouts were recorded.

Embedments/Anchorages
Any expansion anchors or structural embedments that were within five feet of an apparent degradation area were documented should it be necessary to assess the performance of specific anchorages in ASR-affected concrete.

3.2.3 Results

Table 3-1 provides a high level summary of the walkdown results from Reference 9.2.9. The field walkdowns to date have assessed 131 locations including, 106 Phase 1 locations and 25 Phase 2 locations. Of the 106 Phase 1 locations, 50 locations did not exhibit any indications of pattern cracking.

Areas included in the walkdown scope were determined to have a significant risk of developing cracking due to ASR. The key parameter for judging this risk is the exposure of a concrete component to moisture in its current condition. The sources of such moisture exposure are external exposure (i.e. precipitation) and below-grade water ingress. Many plant areas were excluded from the walkdown scope due to a low likelihood of exposure to moisture, (i.e. interior walls, especially above grade) and have a low risk of developing cracking due to ASR. Of the most accessible areas in the walkdown scope deemed to have a significant risk of ASR, about half of them showed no signs of pattern cracking.

Although pattern cracking was noted in many locations throughout the plant, the extent of ASR degradation within a given location is localized and in most areas is minor. The maximum width of an observed crack suspected of being caused by ASR was 0.70 mm. However, the maximum crack width of most ASR-affected areas was ≤0.25 mm. Further, the measured cracking indices are low—the maximum Combined Cracking Index taken from a structural surface is only about 2.5 mm/m.[3] Cracks that appear to be independent of ASR have been evaluated by NextEra Energy and will be followed by the Structural Monitoring Program.

The field walkdowns did not find any locations that require immediate action based on the visual observations gathered.

[3] A cracking index was taken in non-structural grout resulted in a Combined Cracking index of about 3.2 mm/m.

Table 3-1. Summary of Walkdown Results (Reference 9.2.9, Table 2-1)

		Phase 1 (106 Locations)	Phase 2 (25 Locations)
ASR Cracking	Pattern Cracking Present[1]	48	18
	0.0 < CCI < 1.0 mm/m	31	10
	1.0 ≤ CCI < 2.0 mm/m	13	8
	2.0 ≤ CCI < 3.0 mm/m	0	2
	CCI ≥ 3.0 mm/m	1	0
	Max Crack Width	0.70 mm	0.50 mm
Non-ASR Cracking	Yes	53	19
	Max Crack Width	2.50 mm	3.0 mm
Seepage	Active	16	14
	Past	37	24
Popouts	Present	10	0
Supports	Expansion Anchors	45	21
	Structural Embedments	43	16

Notes:

1. The number of locations with pattern cracking present may not equal the sum of the locations in cracking index ranges presented. In some locations where pattern cracking was present, more than one cracking index was performed. Also, the cracking index was not determined for some locations due to access constraints. Cracking indices were also determined for locations where pattern cracking was not conclusively present.

4
Approach for Structural Assessment

The approach for assessing the adequacy of ASR-affected structures at Seabrook Station is based on an extensive review of literature on ASR degradation of concrete, and consultations with recognized experts on ASR and its effects on reinforced concrete structures and equipment anchorages. The discussion in this section reviews how ASR affects concrete and the importance of confinement to provide the technical basis for the approach. The approach of this assessment is then described in detail.

The approach for assessing the adequacy of ASR-affected structures at Seabrook Station relies on structural testing of ASR-affected specimens. Testing of actual structural components affected by ASR provides the best representation of the performance of plant structures. A classical approach would be to determine material properties using cores extracted from plant structures and input these properties into detailed analytical models. However, this approach does not provide an accurate representation of the performance of the actual structures. Once a core is removed from a structure, it loses the confinement provided by reinforcing steel, plant configuration and applied loads (i.e., the "structural context"). As a result, material properties measured using the cores are not representative of the performance of the *in situ* structure. Structural testing, on the other hand provides the best representation possible of the performance of ASR-affected reinforced concrete structures.

4.1 ASR Degradation of Reinforced Concrete Structures

The discussion below draws upon insights from various papers and publications on ASR including References 9.1.5, 9.5.1, 9.5.3, 9.5.4, 9.5.5, 9.5.7, 9.7.4, and 9.7.5.

4.1.1 ASR Mechanism

ASR refers to the reaction between siliceous phases present in some aggregates and hydroxyl ions in the pore solution of concrete. Once the silica is in the solution, it reacts with alkali ions (Na^+, K^+) to create an alkali-silica gel. The gel has a high affinity for water and expands as it absorbs moisture. Expansion of the gel exerts tensile stress on the concrete that can crack the aggregate particles and the cement paste. Typical cracking resulting from ASR is described as "pattern" or "map" cracking (see Figure 4-1) and is usually accompanied by a dark staining around the crack openings.

Figure 4-1. Example of ASR Pattern Cracking on Highway Barriers (Reference 9.5.4)

The extent of ASR degradation and the degradation rate depend on: (1) the reactivity of the specific aggregate(s); (2) the alkali content of the pore solution, which relates to the alkali content of the cement; (3) the presence of moisture to allow alkali migration and to expand the gel which drives the cracking; (4) temperature which impacts reaction rates; and (5) confinement provided by configuration of the structure and the steel reinforcement within the structure.

The cracking may degrade the mechanical properties of the concrete necessitating an assessment of the adequacy of the structures and supports anchored to the structures. The degradation from ASR has been shown to alter the correlations between compressive strength and other concrete properties (e.g., tensile strength, modulus of elasticity) that are inherent in concrete design codes. As noted in References 9.5.4 and 9.5.5, the concrete properties most rapidly affected are the elastic modulus and the tensile strength.

4.1.2 Impact of ASR on Material Properties

The effect of ASR on material properties for unreinforced or unconfined concrete is reasonably well understood. Several publications show that the observed expansion on the surface of unconfined concrete can be correlated to degraded properties such as uniaxial compression, modulus of elasticity, and tensile strength. In fact, Reference 9.5.5 provides lower-bound degraded properties based on measured free expansion in unconfined concrete. The lower bound properties tabulated in Reference 9.5.5 show that compressive strength is a weak function of the observed expansion, while tensile strength and elastic modulus are much stronger functions of the extent of ASR degradation.

4.1.3 Effect of Confinement on ASR Expansion

ASR affects confined concrete structures differently than an unconfined structure. Concrete structures can be confined by one or both of the following: (1) internal reinforcement and (2) externally-applied restraints or stresses. Confinement of a concrete structure limits the ASR expansion and therefore cracking. The effect of confinement is visualized in the following photographs of an ASR-affected, reinforced concrete beam:

- Figure 4-2 shows the midsection of the long side of the beam. This face of the beam is confined by internal reinforcement in both the horizontal and vertical direction.
- Figure 4-3 shows the end face of the same concrete beam. The end face of the beam has minimal internal reinforcement in the same plane as the end face.

Figure 4-2. Midsection of ASR-Affected Beam (Confined Face)

Figure 4-3. End of ASR-Affected Beam (Unconfined Face)

The difference in cracking between the two beam faces is dramatic. The tensile stress created by the ASR expansion in the midsection of the beam is resisted by the internal reinforcement, thereby limiting crack size and maintaining structural integrity.

When the rebar carries the tensile stress exerted by ASR, the core of the concrete, within the rebar cage, is compressed. This effect is similar to the effect of concrete prestressing. In some cases, the prestressing effect of ASR creates a stiffer structural component with a higher ultimate strength than an unaffected member (Reference 9.5.1).

The concrete prestressing effect is only present when the concrete is confined. The concrete prestressing effect is lost when the concrete is taken out of the stress field (e.g., core removed from a wall). A core taken from a confined ASR-affected structure will lose its confinement and no longer represents the context of the structure. Measured mechanical properties from a core taken from a confined ASR-affected structure have limited applicability to the *in situ* performance and only represent the performance of an unconfined or unreinforced structure.

The effect of confinement on ASR-affected concrete is discussed further in Reference 9.5.1.

4.1.4 Performance of ASR-Affected Structures

Various researchers have investigated the performance of structures affected by ASR. These tests have involved both laboratory-prepared specimens and specimens recovered from actual structures, and specimen sizes up to full-scale. Reference 9.5.1 includes a review of these tests focusing on those that are most applicable to reinforced concrete structures similar to those at Seabrook Station. The studies cited therein showed that the structural performance of the specimens was typically better than would have been expected based on calculations using concrete properties measured from cores taken from the structures. This discrepancy is attributable to the effect of confinement as discussed above.

4.1.5 Conclusions

Confinement is a key factor regarding the impact of ASR on reinforced concrete structures. Confinement limits ASR expansion of the *in situ* structure, which reduces the extent of deleterious cracking and the resultant reduction in concrete properties. Given this interplay between an expansive ASR degradation and structural restraint, it is imperative that evaluation of the structural impacts due to ASR focus on structural testing rather than typical materials type testing on cores removed from the structure.

4.2 Structural Assessment Approach

The structural assessment has the following three elements.

- Structural Demand and Seismic Response—ASR may impact the stiffness of a structure, which would alter its dynamic response during a seismic event. This impact is evaluated to assess whether the structural demand is impacted.

- Capacity of Structural Components—ASR degrades the capacity of structures as described earlier in this section.

- Capacity of Anchors—The capacity of embedments and anchors can be impacted by the microcracking and macrocracking associated with ASR.

The potential impact on capacity of a structure or anchor will be considered in conjunction with the potential change in structural demand to provide an integrated assessment.

The structural assessment documented herein relies on available data from testing with reinforced concrete specimens that are affected with ASR. This includes testing to quantify structural performance of ASR-affected concrete structures, and testing to quantify the performance of anchors in ASR-affected concrete. Reference 9.5.1 presented a comprehensive review of the available data on the impact of ASR on reinforced concrete structures, identifying several significant gaps in available data. These gaps relate to shear and reinforcement anchorage.

- Shear capacity of ASR-affected reinforced concrete structures without transverse reinforcement – Most of the available data are based on beams which have reinforcement in all three directions, including the transverse direction. The data available for components without transverse reinforcement used antiquated plain reinforcement (i.e., no deformations) with low yield strength (approximately 30 ksi) and required an extensive retrofit to generate a shear failure (Reference 9.5.10). The modern rebar used in Seabrook Station is markedly different in terms of strength (60 ksi minimum) and deformations (i.e. ribs) on the surface of the bar. The application of this data to the concrete components at Seabrook Station will provide an excessively conservative and potentially misleading conclusion.

- Performance of reinforcement anchorage in ASR-affected concrete without transverse reinforcement – Reinforcement anchorage is most important with regard to moment transfer between rebar at lap splices. The available data on reinforcement are limited to small rebar sizes (#5) (Reference 9.5.8). Further, the available documentation for this testing does not allow a considered assessment of applicability to Seabrook Station.

MPR and FSEL have initiated test programs to address the above gaps. The tests will use large beams to provide a full-scale simulation of portions of structures at Seabrook Station. The testing will include beams with varying levels of ASR degradation from no degradation (control specimens) to levels consistent with that currently observed at Seabrook Station and levels well beyond that observed at the plant.

Reference 9.5.1 also identified a lack of available data on the impact of ASR on embedded anchors (e.g., expansion and undercut anchors). This gap is already being addressed in testing at FSEL subcontracted by MPR. However, the testing to date has used ASR-affected concrete specimens available at FSEL. Future test series will use test specimens fabricated to more closely represent the configuration at Seabrook Station, and will expand the data to cover a range of embedment depths.

The final data from these programs will be incorporated into the long-term assessment of the impacts of ASR. In the interim period, published test data will be used to assess plant structures and data from initial anchor test series will be used to assess the performance of anchorages.

4.3 SUITABLE FOR CONTINUED OPERATION VERSUS DESIGN BASIS

The evaluations herein utilize approaches and criteria that are consistent with those used in operability assessments as opposed to design basis evaluations. This approach is appropriate because the report focuses on the near-term adequacy of concrete structures affected by ASR and attachments in ASR-affected concrete. When test data from the various test programs are available, the effect of ASR on structures and attachments will be reconciled with the plant's design basis analyses.

Acceptance criteria and various code expressions in ACI 318-71 are typically based on lower bound values determined from review of data from myriad tests available in open literature. The evaluations herein consider the margin between the lower bound values used in the code and the expected performance (i.e., the average) of the test data to establish the nominal capacity of the structures. It is noted that the evaluations are still conservative given the inherent conservatism in the load factors and material factors used and in the conservatism in the applied loads.

5

Evaluation of Structural Demand and Seismic Response

This section includes an assessment of the impact of the extent of ASR on the demand and seismic response of safety-related and some maintenance rule structures. This evaluation employs the following approach:

- Identify the types of demand which form the design basis.
- Consider and evaluate the effects of the currently demonstrated extent of ASR on the demand and the seismic response of unreinforced and reinforced concrete structures.

The conclusions of the MPR assessment are compared to those of a detailed study commissioned by NextEra Energy on the effects of the currently demonstrated extent of ASR in the walls of the Containment Enclosure Building.

5.1 Design Basis

The governing design loads of the reinforced concrete structures affected by ASR at Seabrook Station vary by structure and sometimes by elevation within a structure. Some examples include:

- Safe Shutdown Earthquake (SSE) loads – Containment Enclosure Building (CEB).
- Live/Equipment-related loads – Containment Building (CB) – Loss of Coolant Accident (LOCA) pressurization loads (note that the Containment Building at Seabrook Station is protected from the environment by the CEB and an annulus about 5' wide).
- Environmental loads – Below-grade portions of the B Electrical Tunnel (Control Building) and the Residual Heat Removal (RHR) Equipment Vault– Hydrostatic head loads (note that seismic loads are only a small fraction of the load profile for many below-grade areas).

5.1.1 General Seismic Design Characteristics

In the design basis, seismic loads on many reinforced concrete structures at Seabrook Station were determined using a spectral response approach. Using the spectral response method, seismic loads were determined based on the structure's natural frequency, or its inverse, the natural period.

In its simplest form, the natural frequency of a linear dynamic system can be characterized by the following equation (Reference 9.7.6, Sections 3-1 and 3-2):

$$f = \frac{1}{2\pi} \cdot \sqrt{\frac{k}{m}}$$

Where: f = structure's natural frequency (Hz)
k = stiffness of the structure (lbm/sec^2)
m = mass of structure (lbm)

The effect of ASR on the dynamic response of a reinforced concrete structure can be characterized by the simple equation for frequency of vibration given above.

The equation shows that the natural frequency of a structure is proportional to the square root of the structure's stiffness divided by its mass. ASR may affect the stiffness of a reinforced concrete structure, but the mass of the structure is not affected.

5.1.2 Seismic Design Spectra

The horizontal response spectrum for Seismic Category I structures at Seabrook Station is shown in Figure 5-1. Table 3.7(B)-1 of the Seabrook Station UFSAR (Reference 9.6.1) states that, for SSE loads, reinforced concrete structures are designed using 7% of critical damping.

Using the response spectrum method, structural loads are proportional to the response acceleration associated with the structure's natural frequency. Spectral accelerations are represented on one of the non-orthogonal axes represented on Figure 5-1.

Figure 5-1 shows that for natural periods between about 0.10 sec and 0.4 sec (frequencies between 10 Hz and 2.5 Hz), there is very little change in the spectral acceleration. This is the result of peak-broadening for the most-likely ground response spectrum. The maximum response acceleration occurs at a natural period of 0.4 seconds (frequency of 2.5 Hz). Many reinforced concrete structures have a natural period and frequency close to this peak value.

For a reinforced concrete structure with a natural frequency of 2.5 Hz (0.4 second period), Figure 5-1 shows that for 7% damping, the spectral velocity is 16.7 in/sec, and the spectral acceleration is about 262 in/sec^2 or 0.68 g. This is the maximum response acceleration, and is significantly greater than the maximum 0.25g ground acceleration.

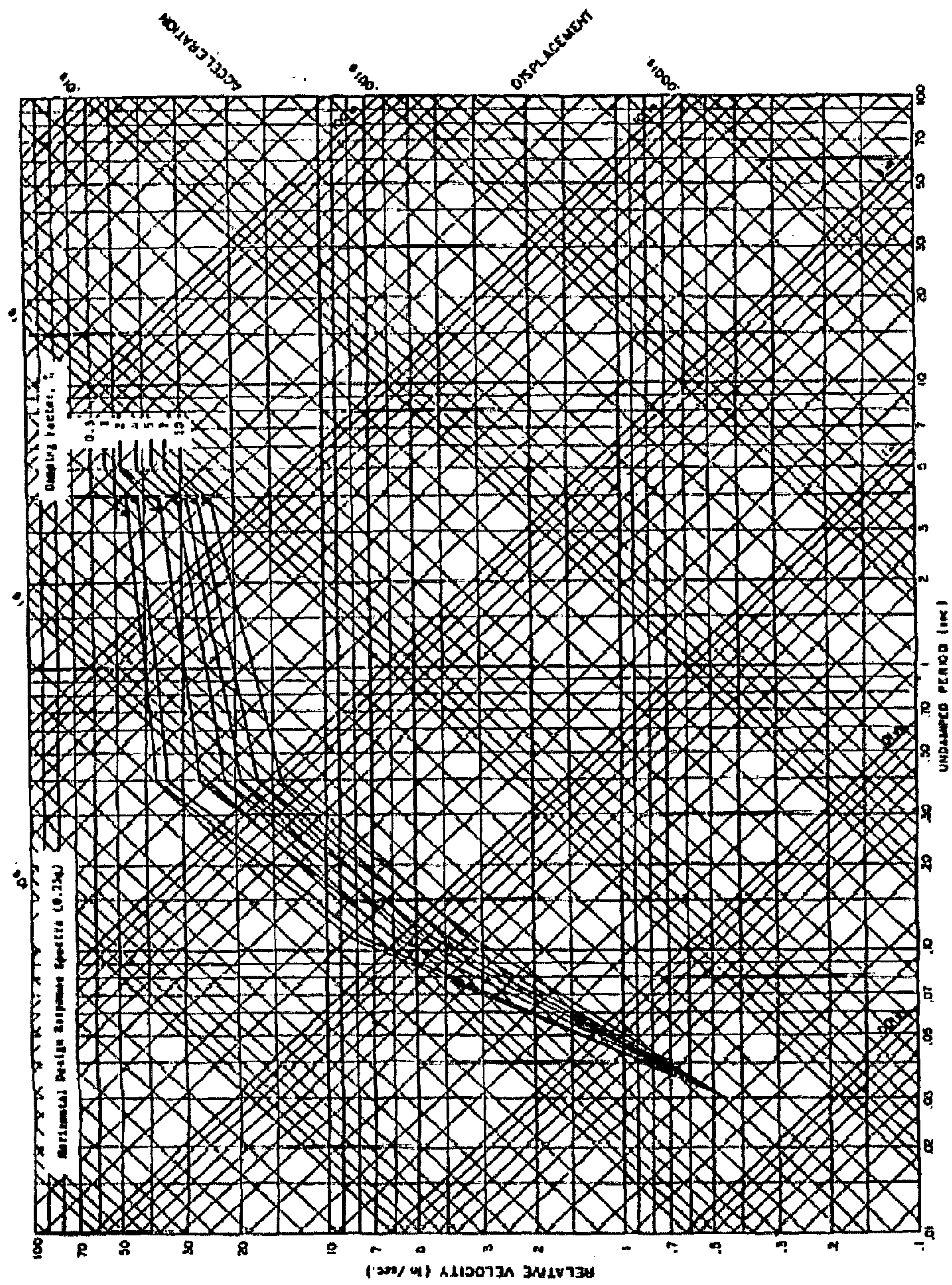


Figure 5-1. Safe Shutdown Earthquake Horizontal Response Spectrum
(Reference 9.6.1, Figure 2.5-43)

5.2 STIFFNESS OF REINFORCED CONCRETE COMPONENTS

The response of a structure to a dynamic load, such as seismic, is proportional to the stiffness of the structure. ASR has been shown to reduce the stiffness of unconfined concrete. The effect of ASR on the stiffness of a reinforced concrete structure is discussed in the context of a generic structure and structures at Seabrook Station.

Mechanical testing of unconfined concrete cores shows that significant degrees of ASR can reduce the modulus of elasticity of unconfined concrete (Reference 9.5.5, Section 4.4). A summary of bounding effects from varying degrees of ASR from mechanical testing of unconfined concrete cores is shown in Table 4 of Reference 9.5.5.

Unreinforced concrete cores subjected to ASR generally contain internal microcracking and macrocracking, leading to reduced strength and stiffness normally associated with ASR. The Institution of Structural Engineers emphasizes the following (Reference 9.5.5, Section 4.4):

> *It is emphasized that the residual strengths and stiffnesses in actual structures will be modified from the figures in Table 4. This is because the concrete in actual structures is generally restrained by adjacent material and is in a biaxial or triaxial stress state. These effects will tend to reduce the damage to the concrete and increase its residual mechanical properties.*

While ASR generally reduces the stiffness of unreinforced concrete, reinforced concrete structures affected by ASR behave differently from unreinforced cores due to confinement of the concrete by the reinforcing bars. ASR has been shown to significantly increase the post-elastic stiffness of reinforced concrete components, at least for concrete structures triaxially confined by reinforcement (Reference 9.5.1, Section 4.3). As shown in Figure 5-2, the stiffness of an ASR-affected reinforced concrete component remains unchanged within the linear-elastic regime. Figure 5-2 shows the much improved strength and stiffness behavior of the component in the non-linear portion of the response. Since there is little difference in the behavior of the ASR-affected concrete and the unaffected control beam in the elastic regime, it is reasonable to infer that natural frequency of both components would remain the same in that portion of the response curve. It is important to note that the overall strength of the ASR-affected component is not compromised.

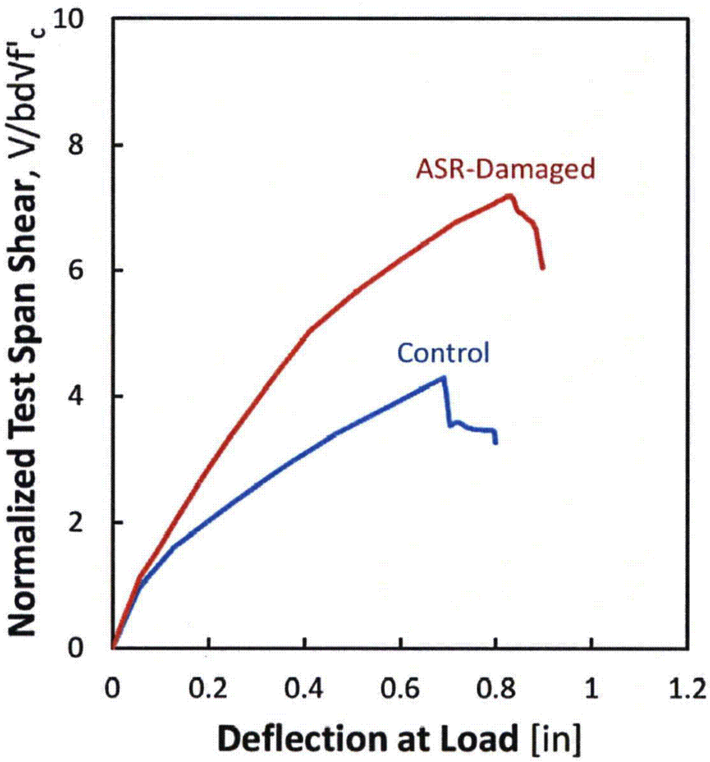


Figure 5-2. Stiffness Testing of ASR-Affected Beams (Reference 9.5.1, Figure 7)

In short, in a case where the benefits of triaxial confinement can be established through the presence of a three dimensional reinforcement cage, there does not appear to be any adverse effect on structural response due to ASR. The apparent increase in stiffness and strength illustrated in Figure 5-2 is a result of the confining effect of the reinforcing steel. The ASR-related expansion of concrete relative to the reinforcement places the concrete in a prestressed-compression condition, which leads to the increased ultimate load shown in Figure 5-2.

Many structural components at Seabrook Station are confined with reinforcement in two-directions and do not include transverse reinforcement. These structural components will likely perform similar to the ASR-affected and control load deflection curves shown in Figure 5-2 in the elastic regime. Full-scale testing will be required to accurately model the effects of confinement on an ASR-affected reinforced concrete component with two-dimensional reinforcement. Such a full-scale testing program is in-progress.

5.3 Study of Containment Enclosure Building

SGH performed a study of the CEB using a visual survey and finite element analyses (References 9.3.1, 9.3.2, 9.3.3, and 9.3.4). The visual survey is documented in Reference 9.3.1. Most of the survey locations were in the CEB, and additional surveys were conducted in each of the following structures: B Electrical Tunnel, the Diesel Generator Building, the EFW Pumphouse, and the RHR Equipment Vault. In addition, digital photographs were taken and a visual condition rating was assigned to each location surveyed. The crack index measurements were obtained using the procedure outlined in Reference 9.5.4. The Overview of Results and Conclusions on the cover page of Reference 9.3.1 states:

> *CI values and typical crack widths at the CEB wall are less than the minimum values (CI value of 0.018 in/yd (0.50 mm/m) and/or crack width of 0.006 in (0.15 mm) specified in FHWA Report HIF-09-004 as indicative of concrete likely undergoing ASR. Fundamental differences between the CEB wall and the basis structures for the FHWA document may compromise the applicability of the FHWA document to the CEB wall when assessing the probability, structural effects and prognosis of ASR.*

Because the majority of the crack index measurements and crack widths observed were less than the minimum criteria given in the FHWA Report HIF-09-004 (Reference 9.5.4), SGH concluded the applicability of FHWA criteria to Seabrook structures was questionable. Instead, SGH developed subjective visual rating criteria to quantify the degree of ASR.

The subjective rating criteria were based on cores obtained near locations where visual ratings were taken. These cores showed variations in concrete properties that were attributed to ASR. The correlation of mechanical properties to the degree of ASR in specific locations was performed in Reference 9.3.2 based on the visual ratings.

The modified concrete properties were used to determine the effects of ASR on the response of the CEB with a dynamic analysis (Reference 9.3.3) and on the demand of the CEB walls with a finite element analysis (Reference 9.3.4). The Overview of Results and Conclusions on the cover page of the dynamic analysis (Reference 9.3.3) states:

> *The maximum acceleration profiles and ISRS are not significantly impacted by the averaged ASR-damaged properties.*

The Overview of Results and Conclusions on the cover page of the finite element analysis (Reference 9.3.4) states:

> *The ASR damage in concrete does not significantly impact the overall forces/moments in the wall.*

Continuing from Section 3.2 (Reference 9.3.4):

> *ASR damage on average does not affect the DCR[4] values in the CEB wall. This behavior is valid both for both OBE and SSE load conditions.*

The dynamic and finite element analyses showed minimal difference in the seismic response and demand on the CEB based on nominal vs. ASR-affected concrete properties.

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[4] Demand to Capacity Ratio

5.4 Conclusions

The impact of ASR on structural demand and seismic response of the reinforced concrete structures at Seabrook Station is negligible. The bases for this conclusion are listed below.

- The shape of the seismic response spectrum makes it unlikely that ASR will increase seismic loads on the reinforced concrete structures at Seabrook Station. There is very little change in the seismic response between frequencies between 2.5 and 10 Hz. Decreases in stiffness at frequencies below 2.5 Hz decrease seismic response.

- The natural frequency of structures is proportional to the square root of the structure's stiffness, thereby reducing the percent change in natural frequency for a given change in material stiffness.

- Typical mechanical property tests that describe the effects of ASR on concrete are based on tests of relatively small, unreinforced concrete cores. These tests show a significant reduction in stiffness (concrete elastic modulus). Tests of full-scale reinforced concrete beams indicate that ASR may have little impact, or potentially may increase stiffness of reinforced members. The triaxial confining effect of concrete reinforcement allows a compressive prestress to develop in the concrete in resistance to ASR-related expansion.

- Design loads on concrete structures generally are based on the sum of several load categories such as dead load, live load, hydrostatic loads and seismic loads. Changes in stiffness associated with ASR only affect dynamic loads such as seismic. While seismic loads are significant in some areas, the fact that ASR does not affect the other loads that contribute to the total reduces the overall impact of ASR on governing load combinations. Changes in elasticity can affect load distributions in redundant structures (e.g., monolithic concrete). The structures at Seabrook Station were analyzed using relatively simple load assumptions and load distributions, so a change in elasticity will not significantly affect individual components.

5.5 Future Actions

A full-scale testing program to quantify the effect of varying degrees of ASR on structural component stiffness (EI) is in-progress. The basis for the conclusion that ASR has a negligible effect on the structural demand and seismic response of the reinforced concrete structures at Seabrook Station will be further justified through the full-scale testing program.

6 Evaluation of Structural Components

This section assesses the impact of ASR on the structural performance of safety-related and some maintenance rule structures. This evaluation employs the following approach:

- Identify the types of structures at Seabrook Station and the related design basis.
- Perform a screening to select a sample of structural components for detailed evaluation. The sample will be biased to areas with significant indications.
- Evaluate the ability of safety-related structural components to perform their design safety function given the currently demonstrated extent of ASR. The evaluation of ASR-affected concrete structures will identify any recommended actions.

6.1 Design Basis

The design of safety-related concrete structures at Seabrook Station is governed by ACI 318-71 (Reference 9.1.1). ACI 318-71 provides load cases for various natural and man-made loads. These loads include, but are not limited to: deadweight, live loads, hydrostatic head, wind, tornado missile and seismic.

The individual concrete components are integrally cast, creating monolithic structures. Individual concrete structural components can be divided into the following categories:

- Load-Bearing Walls and Columns: These reinforced concrete elements are constructed in the vertical direction. Vertical load is transferred through the element in compression to the base mat. Horizontal load is resisted through flexure and/or shear and transferred to the base mat.
- Floor Slabs, Base Mats and Beams: These reinforced concrete elements are constructed in the horizontal direction. Floor slabs and beams resist applied loads though flexure and shear and transfer the load to vertical elements (walls and columns). Base mats distribute concentrated loads from vertical elements and some applied loads onto the subgrade through flexure and shear.

6.1.1 General Design Information

Specified concrete strengths vary among structures at Seabrook Station. Concrete strength of 3,000 psi was specified for most structures, but 4,000 psi was specified for a few structures and 5,000 psi was specified for one maintenance rule structure (Reference 9.6.3). Statistical evaluations of compression tests from original construction (Reference 9.2.11) revealed that the average compressive strength of concrete specified for 3,000 psi was 4,359 psi with no individual cylinder test showing a compressive strength value less than 3,500 psi.

The thickness of load-bearing walls are typically two feet or greater. Columns are used occasionally either stand-alone or as part of a load-bearing wall, and are typically four feet square or larger. Floor slabs are at least one foot thick, usually greater. Beams are used in very few locations. Base mat thicknesses usually vary between four and six feet.

Steel reinforcing bars conform to ASTM A615 (Grade 60 deformed bars) (Reference 9.6.2). Typical bar sizes for safety-related structures are #8 to #11 except for containment which utilizes larger bars. Reinforcing bars are typically placed in two-directional mats with one mat near each concrete face. The spacing between individual bars typically range from 6 to 12 inches. Clear concrete cover for rebar is typically 2 inches for internal faces and 3 inches for external faces. Transverse reinforcement (i.e. reinforcement provided through the wall thickness) is only provided in limited applications.

6.1.2 General Design Approach

As discussed in Section 1.2, the station layout minimizes the site footprint and height of the structures above grade. This layout resulted in a station that is very compact and contains more below grade areas than is typical. Many structures are only separated by a three-inch thick isolating material, permitting them to act independently in a seismic event. This small gap between many of the safety-related structures does not permit the external assessment of many walls (above or below-grade).

Design of the below-grade portion of the station structures is usually governed by the large hydrostatic load instead of seismic and equipment loads. External wall designs tend to be governed by flexure or out-of-plane shear. Internal walls act as braces for the external walls and their designs are usually governed by in-plane shear. Many walls are designed to carry a high load eccentricity (i.e. high bending moment relative to vertical load) and their loading more closely resembles that of a beam instead of a column. The design of above grade walls are typically governed by equipment loads (large equipment or pipe whip) or natural loads such as seismic or tornado missile.

6.2 SCREENING OF STRUCTURAL COMPONENTS

The reinforced concrete structures are screened to identify structures or portions of a structure that require a more detailed evaluation. The screening uses the observed severity of degradation based on the Combined Cracking Index (CCI)[5] and maximum ASR crack width from the walkdowns (Reference 9.2.9), and guidance from published studies (e.g., References 9.5.4, 9.5.5, and 9.5.6) to disposition some structures or portions of structures as having negligible to minimal structural degradation.

6.2.1 Currently Available Screening Methods

Several published studies describe screening methods to determine when structural evaluations of ASR-affected concrete are appropriate and how to prioritize such evaluations. Three screening methods from published studies are briefly summarized below. These three screening methods will be combined to form the basis of the screening criteria for the structures at Seabrook Station. While these screening methods are based on lightly or unreinforced concrete structures, they are useful in the absence of criteria directly relevant to the highly-reinforced concrete structures used in nuclear generating facilities.

- The Institution of Structural Engineers (U.K.) publication *Structural Effects of Alkali-Silica Reaction* (Reference 9.5.5, Sections 6.3.2 and 8.2) describes a screening method for ASR-affected concrete using five categories based on studies of unreinforced structures as outlined below:
  - Category I: Expansions on the order of 0.4 mm/m are of no concern even if ASR has been identified petrographically as they occur in the normal service of concrete unaffected by ASR. Expansions up to 0.6 mm/m will only marginally impact strength.
  - Category II: Expansions in the range of 0.6 to 1.0 mm/m have an impact on some concrete characteristics such as tensile strength, but will only have a marginal impact on highly reinforced structures.
  - Category III: Expansions in the range of 1.0 to 1.5 mm/m should have a detailed appraisal with consideration to potential capacity reductions.
  - Category IV: Expansions in the range of 1.5 to 2.5 mm/m require a detailed appraisal with consideration to potential capacity reductions.
  - Category V: Expansions of 2.5 mm/m or greater should be subject to special study, testing and monitoring.
- The U.S. Department of Transportation – Federal Highway Administration publication *Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in*

[5] The Combined Cracking Index (CCI) is the average of the horizontal and vertical Cracking Indices. Cracking Indices are further discussed in Section 3.2.2.

Transportation Structures (Reference 9.5.4, Section 4.2.4) identifies cracking criteria based on studies of unreinforced ASR-affected structures. More detailed investigations are justified if expansions of 0.5 mm/m or individual cracks of 0.15 mm or greater are identified.

- Oak Ridge National Laboratory publication *In-Service Inspection Guidelines for Concrete Structures in Nuclear Power Plants* (Reference 9.5.6, Section 5.4.6) identifies cracking criteria for ASR-affected concrete using four categories based on a study of lightly reinforced concrete beams with undeformed reinforcement. Reference 9.5.6 indicates that structures in categories 1 and 2 have not likely been significantly damaged and structures in categories 3 and 4 require structural evaluation. Those categories are explained below:
  - Category 1: Crack widths up to 0.2 mm.
  - Category 2: Crack widths in the range of 0.2 to 1.0 mm.
  - Category 3: Crack widths in the range of 1.0 to 2.0 mm.
  - Category 4: Crack widths greater than 2.0 mm[6].

6.2.2 Selection of Screening Criteria for Structural Components

In the absence of studies more relevant to the reinforced concrete design and detailing used at Seabrook Station, the selected screening criteria in Table 6-1 utilize a combination of all three of the previously described criteria. It is recommended that these screening criteria are updated when more relevant studies are available.

Table 6-1. Criteria for Screening ASR-Affected Areas

Recommendation for Individual Concrete Components	Combined Cracking Index (CCI)	Individual Crack Width
Structural Evaluation	1.0 mm/m or greater	1.0 mm or greater
Quantitative Monitoring and Trending	0.5 mm/m or greater	0.2 mm or greater
Qualitative Monitoring	Any area with indications of pattern cracking or water ingress	

Note: The criteria related to expansion due to ASR are expressed in terms of CCI to be consistent with the field walkdown results.

[6] Due to a typographic error, this value was reported as 0.2 mm in Reference 9.5.6.

6.2.3 Implementation of Screening Criteria for Structural Components

The results of the screening of ASR-affected areas recommended for structural evaluation are provided in Table 6-2. The screening criteria are applied on the results of field walkdowns (Reference 9.2.9).

Table 6-2. Results of ASR-Affected Area Screening for Structural Evaluation

Criteria	Screened Out	Screened In
Combined Cracking Index (CCI): 1.0 mm/m or greater	107	24
Individual Crack Width: 1.0 mm or greater	131	0

Note: A few cracks with a width of 1.0 mm or greater were identified during the field walkdowns, but none of these were dispositioned as caused by ASR. The evaluation of these cracks are covered by the Seabrook Station Structural Monitoring Program.

The eleven areas selected for detailed evaluation are identified in Table 6-3. The selection is a sample of the areas screened in from Table 6-2. The sample of the screened-in areas is biased to include the areas with the highest combined cracking index. Any area with a CCI of 1.5 mm/m or greater was selected for the sample[7]. The largest CCI in the selection is about 2.5 mm/m. The selected areas in Table 6-3 include areas previously identified by NextEra Energy (References 9.7.7 and 9.7.8) as areas of concern.

[7] In one area, a CCI was taken in non-structural grout and exceeds 1.5 mm/m. This area was not selected for detailed evaluation because the area affected appears to be localized to non-structural grout.

Table 6-3. ASR-Affected Areas Selected for Detailed Evaluation

Structure	Elevation	Room Number	Structural Components of Concern	Reference Calculation
RHR Vault	(-) 61' up to (-) 16'	Various	All Walls	PB-30
Emergency Feedwater Pumphouse	(-) 26' up to grade (20')	EFST1	North and East Walls	EF-4
Electrical Tunnel 'B'	(-) 20' up to 20'	CBST1	North, East, and West Walls and Floor Slab	CD-20
RCA Tunnel	0' & 5'	Unit 1 Tunnels	NE Wall @ El. 0' & All Cored Walls	SG-1, CD-10, WB-69, WB-82
Diesel Generator Building	(-) 16'	DG102	East Wall	CD-18
Primary Auxiliary Building	(-) 6'	PB205	South Wall, East Wall (South Portion)	PB-20, WB-82
Primary Auxiliary Building Mechanical Penetration	(-) 34'	MF102	North Wall, Column near NE corner of room	EM-31
Electrical Tunnel 'B'	(-) 20'	EF101	North, South Walls and Floor Slab	EF-4, EF-11
MS/FW Pipe Chase (East)	Above Grade (>20')	Exterior	East Wall	EM-19
Cooling Tower	Above Grade (>20')	Unit 1 Exterior	South Wall & North Pipe Chase Bump-out	CT-53, CT-28
Service Water Pumphouse	Above Grade (>20')	Exterior	North Wall of SW Bump-out & South Wall	CW-29

6.3 Detailed Component Evaluations

The detailed evaluations reviewed the relevant design-basis calculations to assess the current margin, considering the worst-case effects of advanced ASR degradation and conservatisms in the ACI code as documented in code committee reports.

The detailed component evaluations:

- Documented the margin in the design basis calculation for each component in the selected ASR-affected areas.
- Identified the evaluations within the calculation that are adversely affected by ASR degradation of the concrete.
- Identified analysis options that could be employed to increase the margin in evaluations that are adversely affected by ASR degradation of the concrete. The review did not address margin that could be gained by methods outside of analysis space, such as anticipated full-scale structural testing.
- Estimated the amount of unnecessary conservatism that could be removed if the recommended analysis options were pursued.
- Identified areas that would likely not meet acceptance criteria after applying the potential strength reductions due to ASR degradation, even with unnecessary analysis conservatisms removed.

This review is documented in MPR Calculation 0326-0058-63, which is included as Appendix A of this report.

6.3.1 Screening Criteria for Detailed Component Evaluations

The detailed evaluations of structural components focus on the limit states of reinforced concrete design affected by ASR. Table 6-4 compares these limit states with the effects of ASR as documented in literature (Reference 9.5.1). Table 6-4 includes assessments of whether or not a given limit state is a concern for Seabrook Station structures. The rationales for these judgments are provided as footnotes to Table 6-4. Conclusions from Table 6-4 are:

- ASR has potential to reduce the ability of concrete to develop the full strength of reinforcement at locations of reinforcement lap splices and at locations of reinforcement straight bar embedment (i.e., embedments without hooks) in areas that a three-dimensional reinforcement cage is not provided. Sufficient length is required in the reinforcement lap splice length and in the embedment length to fully develop the strength of the reinforcing steel.
- ASR has the potential to reduce the ability of the concrete to resist out-of-plane (one-way) shear loads in areas that a three-dimensional reinforcement cage is not provided. One-way shear also envelopes in-plane shear. In-plane shear primarily resisted by flexural

reinforcement and is more sensitive to the affects of ASR due to its potential effects on reinforcement development instead of shear in the concrete.

- ASR has no significant effects on flexure, one-way shear with transverse reinforcement, two-way shear, and reinforcement anchorage with transverse reinforcement.
- ASR effects on compression are not a consideration for the Seabrook Station structures.

Table 6-4. Limit States Considered for Effect of ASR

Limit State		Lower-Bound Effect of ASR (Reference 9.5.1)	Concern for Seabrook Structures?
Axial Compression		Moderate loss of strength (up to 18% loss)	No[1]
Flexure		No significant loss of strength or stiffness (up to 7% loss)	No[2]
One-Way Shear	with transverse reinforcement	No significant loss of strength or stiffness (more than 16% gain)	No
	without transverse reinforcement	High variability among similar specimens (up to 25% loss)	Yes
Two-Way Shear		No significant loss of strength or stiffness (up to 9% loss)	No[2]
Reinforcement Anchorage	with transverse reinforcement	No significant loss of strength or stiffness (up to 10% loss)	No[2]
	without transverse reinforcement	Significant loss of strength (40% loss)	Yes

Notes:

1. The effect of ASR on axial compression is a concern for columns or load-bearing walls with high compression relative to the applied flexure loads, i.e., the concrete compression controlled region of the bending moment and axial load interaction diagram. Review of the components in the sample of ASR-affected components did not identify any compression elements that were compression controlled. All compression components reviewed are controlled by high load eccentricity or the reinforcement tension controlled region of the interaction diagram.
2. These losses are negligible when examined in the context of the normal strength variation tolerated within reinforced concrete construction (Reference 9.5.1). It is reasonable to use no loss of strength for this limit state for determining operability.

Based on Table 6-4, the limit states of one-way shear without transverse reinforcement and reinforcement anchorage without transverse reinforcement are of concern to Seabrook structures affected by ASR.

The screening criteria used to evaluate whether out-of plane shear and reinforcement anchorage are a potential concern for a given location are developed below. These screening criteria consider the strength reductions from Table 6-4 and conservatism in ACI acceptance criteria as discussed in Reference 9.5.2.

Out-of-Plane Shear

Potential strength reductions of up to 25% for out-of-plane shear in ASR-affected concrete are identified in Table 6-4. This potential reduction is based on testing of 5" x 3" concrete prisms without transverse reinforcement (Reference 9.5.9). The results of the testing had high variability, with a maximum enhancement of shear strength of 12% and a maximum reduction of 25%. The potential out-of-plane shear strength reduction of 25% is conservative because it is the maximum reported reduction in the test program.

Conservatism in the calculated capacity for out-of-plane shear strength in the ACI Code equations of approximately 50% is documented in ACI Code Committee reports per Reference 9.5.2. This conservatism is applicable for elements with two-dimensional rebar, i.e. no transverse reinforcement. The lower bound of the data is what forms the basis for ACI 318-71 code requirements. The use of average values is appropriate for the purposes of evaluation of operability. The combination of conservatism in ACI 318-71 with regard to expected performance and the maximum potential performance reduction due to the effect of ASR is that there is no net reduction in shear capacity relative to that calculated using ACI 318-71 for components as thick as two feet.[8] For components thicker than two feet, the reduction in shear strength is expected to be 25% and no credit is taken for the conservation identified in Reference 9.5.2. The criterion of 25% for out-of-plane shear is the potential reduction for components thicker than two feet. This criterion is used in the detailed evaluations to differentiate between evaluations that are of concern and evaluations that are not of concern.

Reinforcement Lap Splices and Anchorage

Potential strength reductions of 40% for reinforcement lap splices in ASR-affected concrete are identified in Table 6-4. The potential strength reduction of 40% is the average strength reduction reported, which is appropriate for an operability assessment. This potential reduction is based on reinforcement pullout testing in concrete without transverse reinforcement reported in Reference 9.5.8. The potential lap splice strength reduction of 40% is likely conservative because the reinforcement pullout testing targeted a weaker failure mode for a component with a low or moderate concrete cover to bar diameter ratio. This weaker failure mode is described in ACI Code Committee reports (Reference 9.1.2). In Figure 6-1, the experimental study used test method (a) while structural performance is best represented by test method (d). ACI 408R-03 (Reference 9.1.2) states:

> *The pullout specimen (Fig. 1.6(a)) is widely used because of its ease of fabrication and the simplicity of the test. ... This specimen is the least realistic of the four shown in Fig. 1.6 because the stress fields within the specimen match few*

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[8] This conclusion applies when the evaluation is based on a design f'_c of 3,000 or 4,000 psi, as appropriate to the building being reviewed. This conclusion does not apply when the value of f'_c in the calculations is based on test data. For these cases, the reduction in shear strength is expected to be 25% and no credit is taken for the conservatism identified in Reference 9.5.2.

cases in actual construction. ... Thus, the use of pullout test results as the sole basis for determining development length is inappropriate and not recommended by Committee 408. ... Beam anchorage and splice specimens shown in Fig. 1.6(c) and (d), respectively, represent larger-scale specimens designed to directly measure development and splice strengths in full-size members.

The experimental study in which 40% anchorage strength reduction was measured employed #5 bars. Directly applying those test results to the anchorage performance of much larger reinforcing bars (generally #8 to #11 for safety-related structures other than containment) is conservative.

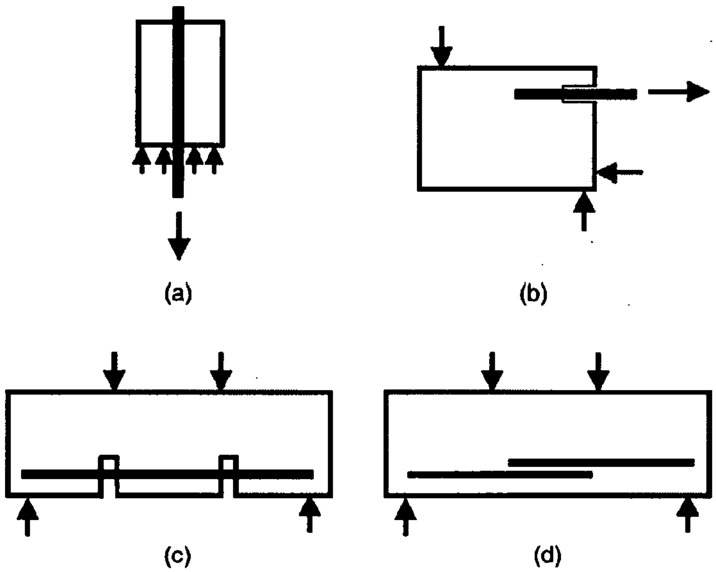

Figure 6-1. Reinforcement Development Test Methods
(Reference 9.1.2, Figure 1.6)

Conservatism in the calculated capacity for reinforcement lap splice strength in the ACI Code equations of 23% is documented in ACI Code Committee reports per (Reference 9.5.2). This conservatism is applicable for components with two-dimensional rebar, i.e. no transverse reinforcement. In this regard, "conservatism" is established by considering the average of the test-to-prediction ratios. The lower bound of the data is what forms the basis of code calibration. The use of average values is appropriate for the purposes of evaluation of operability, until more appropriate data such as full-scale structural testing are available.

Based on References 9.5.1 and 9.5.2, a criterion of 17% is justified as the potential reduction in strength of reinforcement lap splices and reinforcement embedments due to ASR.[9] The 17%

[9] The criterion of 17% is the arithmetic sum of +23% and -40%. The relevant ACI 318-71 equations are not linear and can require an iterative approach. A scoping analysis determined that using the sum of the two considerations is more conservative than comparing the two considerations through the ACI design equations (see Appendix A of this report).

criterion is applicable to evaluations that consider the specified compressive strengths of concrete. In cases where the actual compressive strength was used the 23% code conservatism is not applicable and a 40% criterion is used. The 23% code conservatism is also not applicable to reinforcing bar sizes #6 and smaller and the 40% criterion is used in evaluations crediting these smaller bar sizes. These criteria are used in the detailed evaluations to screen between evaluations that are of concern and evaluations that are not of concern.

Lap splice length and embedment length are important to three types of evaluations: (1) reinforcement to carry bending moments, (2) reinforcement to carry in-plane shear loads, and (3) for minimum flexural reinforcement requirements.[10] These are the evaluations that are flagged in the review for further scrutiny.

6.3.2 Scope of Detailed Evaluations

A targeted approach was used for the detailed evaluations. The detailed evaluation process is described below:

- Identify the evaluations within the calculation that address design of the concrete component to carry design basis loads or address minimum required reinforcement in the ASR-affected area.

- Document the calculated margin in the evaluation relative to the code requirement. The margin is expressed as the percentage: Margin = 100% * ((Capacity – Demand) / Capacity. The capacity or demand may be expressed as a force, moment, stress, or rebar area, dependent on how the information was presented in the calculation. The capacity is calculated using the provisions in ACI 318-71. It is not the margin to failure as there is additional margin inherent in ACI 318-71.

- Identify the evaluations for which ASR is a concern. These are the evaluations which meet the following criteria:

  - The evaluation of a wall/slab with a thickness exceeding two feet for which the shear margin in the design basis calculation is less than 25%.

  - The evaluation credits reinforcement for flexure (with or without axial compression or tension) or in-plane shear. Minimum required reinforcement evaluations for flexure and in-plane shear per the limits prescribed in ACI 318-71 are also considered.

  - The wall/slab being evaluated has reinforcement lap splices or reinforcement straight bar embedment (not including the length of straight bar embedment provided with a standard hook to achieve the required development length).

[10] The review assures that lap splices and anchorage for minimum reinforcement are adequately sized, considering possible degradation in strength of lap splices or anchorage from ASR. For walls, the minimum reinforcement is primarily for shrinkage, thermal expansion, and serviceability concerns. If the splices are not appropriately sized to carry these loads, the splices could be compromised and the splices could not then carry design basis loads.

- Identify conservatisms in the calculation with potential to increase the margin in the evaluation or alleviate the ASR degradation concern. The potential conservatisms that were considered are:

  - Eliminating overly conservative simplifying assumptions, e.g., calculating a wall bending moment as a two-way slab rather than a one-way slab where wall aspect ratios permit such analysis.

  - Eliminating unnecessary levels of conservatism in the calculation of the applied loads.

  - Using a more sophisticated analysis method, e.g., a finite element analysis to more accurately calculate the distribution of load. Simple finite element models were prepared to estimate the potential gain with this approach.

  - Taking credit for the actual amount of reinforcement in the wall/slab if this is greater than the amount of reinforcement required to be in the wall/slab in the calculation.

  - Taking credit for adjacent reinforcing steel lap splices that are staggered rather than aligned.

  - Taking credit for a reduction in required splice length when the lap splice is in a low stress area.

  - Using alternate capacity equations from the ACI Code.

  - Determining if the area of interest is affected by ASR indications based on the walkdown results.

  The options considered to improve the calculation margin are generally related to the analysis and the actual construction details. Other potential sources of conservatism are deemed to be outside the scope of this review.

- Estimate the anticipated margin from the methods described. This is the margin that might be obtained with a reanalysis. The estimate is based on a scoping evaluation and is provided for information.

- Identify the evaluations for which ASR is a potential operability concern, taking credit for the potential margin increase that could be obtained from a reanalysis. The evaluations that are a concern are those that meet the screening criteria for the detailed component evaluations for which the anticipated margin is less than the applicable potential strength reduction criteria.

As discussed, for cases where an evaluation did not have sufficient margin to accommodate ASR concerns, an estimate was made of the margin that could be recovered from the evaluation by removing unnecessary levels of conservatism.

The detailed component evaluations focused on the aspects of interest for the ASR evaluation. Although the scope was not to verify the comprehensiveness of the design basis calculations, several calculational deficiencies were identified. These were reported to NextEra Energy for inclusion in the NextEra Energy Corrective Action Program.

6.3.3 Results

The detailed evaluations addressing eleven areas and the 143 specific evaluations are documented in MPR Calculation 0326-0058-63, which is included as Appendix A of this report. A summary of the review results is provided in Figure 6-2 and Table 6-5. There are 15 evaluations in the eleven areas in which the margin in the calculation is not sufficient for the potential degradation of the concrete by ASR, even with reanalysis to remove some unnecessary levels of conservatism. The specific evaluations with insufficient margin are identified in Table 6-6. There are 128 evaluations in the 11 areas that were shown to have sufficient margin (either documented in the design basis calculations or after potential removal of unnecessary conservatism included therein) to accommodate potential degradation of the concrete by ASR. There are 32 evaluations that were shown to have sufficient margin after potential removal of unnecessary conservatism in those evaluations. In particular, the west wall of CBST1, the limiting evaluation for out-of-plane shear in the B Electrical Tunnel, was shown to have sufficient margin after potential removal of unnecessary conservatism.

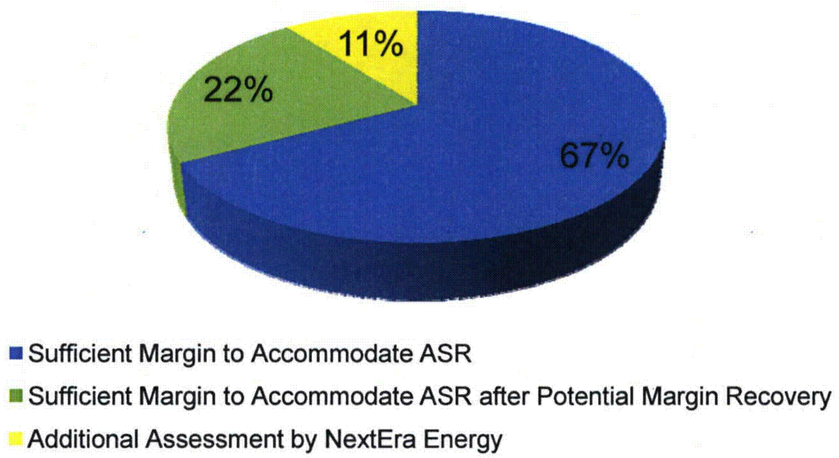

Figure 6-2. Summary of Detailed Evaluation Results for Selected ASR-Affected Areas

These results are based on a review of the design basis calculations and recovery of margin that is available in analysis space. The results do not address margin that can potentially be recovered through other avenues, such as from the planned full-scale structural testing.

The potential conservatism in the structural calculations is an estimate based on scoping evaluations. MPR provided informal checks of these estimates to assure they were reasonable. These scoping calculations are not included in this calculation and the margin recovery estimates are not QA results. The identification of conservatism is an estimate of the likely margin that could be obtained if the design basis calculation were revised.

Table 6-5. Summary of Detailed Evaluation Results for Selected ASR-Affected Areas

Structure	Evaluated Areas with ASR	Number of Evaluations	Evaluations of Concern Before Potential Margin Recovery	Evaluations of Concern After Potential Margin Recovery
RHR Vault	All Walls, -61 ft. to -16 ft.	30	10	1 – See Table 6-6
Emergency Feedwater Pumphouse, EFST1	North and East Walls, -26 ft. to grade (20 ft.)	13	4	1 – See Table 6-6
Electrical Tunnel 'B', CBST1	North, East, and West Walls and Floor Slab, -20 ft. to 20 ft.	11	6	No Evaluations of Concern for ASR
RCA Tunnel, Unit 1 Tunnels	NE Wall @ Elev. 0' & All Cored Walls	5	3	3 – See Table 6-6
Diesel Generator Building, DG102	East Wall, -16 ft.	12	3	1 – See Table 6-6
Primary Auxiliary Building, PB205	South Wall, East Wall (South Portion), -6 ft.	6	0	No Evaluations of Concern for ASR
Primary Auxiliary Building Mechanical Penetration, MF102	North Wall (Column near NE corner of room), -34 ft.	2	0	No Evaluations of Concern for ASR
Emergency Feedwater Pumphouse, EF101	North and South Walls and Floor Slab,[11] -20 ft.	2	0	No Evaluations of Concern for ASR
MS/FW Pipe Chase (East), Exterior	East Wall, above grade (> 20 ft.)	7	2	No Evaluations of Concern for ASR
Cooling Tower, Unit 1 Exterior	South Wall & North Pipe Chase Bump-out, above grade (> 20 ft.)	44	19	9 – See Table 6-6
Service Water Pumphouse, Exterior	North Wall of SW Bumpout & South Wall, above grade (> 20 ft.)	11	0	No Evaluations of Concern for ASR
	Total	143	47	15

[11] The floor slab for EF101 was not included in this review. The design basis calculation was not available.

Table 6-6. Evaluations of Possible Concern for ASR-Affected Areas

Wall	Evaluation	ASR Effect	Amount of Margin Required to Accommodate Potential Effects of ASR Degradation (%)
RHR Vault, Various Rooms			
EL. (-) 45' 4' Ext. Wall	Out-of-Plane Shear	Concrete Shear Capacity Reduced 25%.	1%
Emergency Feedwater Pumphouse, Room EFST1			
East	Vertical Reinforcement for In-Plane Moment El. 0' to 27'	Embedment & Splice Length Increased 17%	11%
RCA Tunnel			
NE Corner of Tunnel	Vertical Reinforcement for Flexure and Compression	Embedment & Splice Length Increased 40%	22%
NE Corner of Tunnel	Horizontal Reinforcement for Shear	Embedment & Splice Length Increased 40%	1%
West Wall (Control Bldg) - Core Bore RCAW-1&2	Flexure and Tension	Embedment & Splice Length Increased 40%	22%
Diesel Generator Building, Room DG102			
East	Flexure	Embedment & Splice Length Increased 17%	7%
Cooling Tower			
South, El. 32' to 39', Cols. A-D	Horizontal Reinforcement for In-Plane Shear, Bending, and Tension	Embedment & Splice Length Increased 17%	3%
South, El. (-) 8' to 21'	Out-of-Plane Shear	Concrete Shear Capacity Reduced 25%	18%
South, El. 21 to 45'	Vert. Reinforcement for Bending	Embedment & Splice Length Increased 17%	19.5%
South, El. >50', Cols. D-K	Vert. Reinforcement for Bending and Tension	Embedment & Splice Length Increased 17%	6 areas ranging from 6% to 12%

6.3.4 Potential Concerns in ASR-Affected Areas

Considering the conservative manner of our approach for the evaluation of ASR-affected structural components, the 15 evaluations that appear to have insufficient margin to accommodate the potential effects of advanced levels of ASR degradation are not unsuitable for continued service. The conservative aspects of our approach for our evaluations are summarized below.

- Potential strength reductions of 40% for reinforcement lap splice and embedment in ASR-affected concrete are not truly representative of the expected performance of these reinforcement limit states.
  - While the study producing an average strength reduction of 40% was the most relevant study for the reinforcing anchorage limit state without transverse reinforcement, the ACI Technical Committee 408 stated in its report that the method used in the study is "inappropriate and not recommended."
  - The test used reinforcing steel significantly smaller (#5 bars) where the structures at Seabrook Station typically use #8 bars and larger for safety-related structures.
  - While the level of ASR in the reinforcement pullout study was not documented well, the tests were run at an advanced stage of ASR degradation. Seabrook Station does have indications of ASR, but it is not at an advanced state.

  The multiple conservatisms apparent in the detailed evaluation approach, coupled with the strong statement published by the ACI Committee 408 on the suitability and reliability of rebar pullout testing as the basis for the strength of reinforcement anchorage in concrete suggest that there is significant uncertainty in the screening criterion applied to the design basis calculations.

- Potential strength reductions of 25% for out-of-plane shear are not 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%. The average impact is a reduction of 6%, which is within the available margin for all areas.
  - The shear capacity reduction due to ASR of 25% is based on a small-scale test using 5-inch x 3-inch beams. It is well known that shear phenomenon does not scale well.
  - While the level of ASR in the out-of-plane shear capacity study was not documented well, the tests were run at an advanced stage of ASR degradation. Seabrook Station does have indications of ASR, but it is not at an advanced state.

  Therefore, the reduction in shear capacity due to ASR is likely less than the 25% used in the screening, particularly in view of the current state of ASR at Seabrook Station.

- The scoping evaluations used a few simple methods to identify potential margins that could be recovered in the design basis calculations. The scoping evaluations did not employ all methods by which to recover margins from over-conservatisms in the calculations.

Based on the points above, there is reasonable assurance that the structures are adequate to perform their design function for an interim period.

Test programs have been initiated to evaluate the impact of ASR on shear capacity and performance of reinforcement anchorages using full-scale beams. The full-scale beams will replicate key features of the B Electrical Tunnel. This method of determining adequacy is essentially proof testing and is deemed a "more precise method" by the ACI code. The full-scale tests will provide a definitive assessment of the nominal margin inherent in the design and any apparent strength reductions due to various degrees of ASR.

NextEra Energy has performed a supplemental assessment to demonstrate adequate margin for the 15 evaluations that initially appeared to have insufficient margin to accommodate ASR. Their evaluation focused on conservatism in the demand (i.e. loads and load factors).

6.4 CONCLUSIONS

There is reasonable assurance that the structural components are adequate to perform their design function for an interim period. The bases for this conclusion are listed below:

- ASR pattern cracking can be observed in many areas within Seismic Category I structures and Maintenance Rule structures, but only a limited portion of these areas have sufficient ASR degradation to merit detailed evaluation.

- The eleven locations selected for detailed evaluation were biased to include the areas with the highest Cracking Indices. Of the 131 locations evaluated during the field walkdowns, only 24 exceeded our screening criterion of a Combined Cracking Index of 1.0 mm/m.

- The detailed evaluations of these eleven areas focused on limit states for which available data indicated that there is a potential decrease in capacity due to ASR: out-of-plane shear, and reinforcement lap splices and anchorage.

  - Out-of-Plane Shear—Available data from scale tests indicate that ASR can potentially reduce shear capacity by up to 25%. However, ACI 318-71 includes approximately 50% margin on the shear capacity for components up to 2 feet thick, but lesser margin for components thicker than 2 feet.

    - For components up to 2 feet thick, ASR should not degrade shear capacity below that calculated from ACI 318-71 as the margin inherent in the code exceeds the maximum reduction in shear capacity.

    - For components greater than 2 feet thick, ASR may degrade shear capacity up to 25% below that calculation from ACI 318-71.

- Reinforcement Lap Splices and Anchorage—Available data from rebar pullout tests, an outdated and unreliable test method, indicate a 40% strength reduction for lap splices in ASR-affected concrete. However, there is approximately 23% conservatism in the ACI 318-71 equations for lap splice strength. Therefore, ASR could decrease lap splice strength about 17% relative to that calculated using ACI 318-71 and specified compressive strength. For cases where the actual concrete compressive strength was credited, the 23% conservatism in the ACI 318-71 equations for lap strength cannot be credited as part of this conservatism derives from the difference between specified and actual compressive strength.

- For the eleven areas subjected to detailed evaluations, a total of 143 evaluations were assessed to determine if there was sufficient margin to accommodate ASR. Of these 143 evaluations, 47 (33%) do not have sufficient margin based on the margin documented in the Seabrook Station calculation. However, after exploring means for potentially recovering margin, only 15 of the 143 evaluations (10%) appear to have insufficient margin to accommodate ASR

- The multiple conservatisms apparent in the detailed evaluation approach, coupled with the strong statement published by the ACI Committee 408 on the suitability and reliability of rebar pullout testing as the basis for the strength of reinforcement anchorage in concrete, suggest that there is significant uncertainty in the screening criterion applied to the design basis calculations.

- Potential strength reductions for out-of-plane shear are not representative of the expected performance of the walls at Seabrook Station. Available data on shear capacity reduction due to ASR are based on small-scale testing—some as small as 5-inch x 3-inch beams. It is well known that shear phenomenon does not scale well.

It is noted that NextEra Energy performed supplemental assessments to disposition the 15 evaluations which did not initially appear to have sufficient margin to accommodate ASR.

6.5 FUTURE ACTIONS

Test programs have been initiated to evaluate the performance of two key limit states in the absence of transverse reinforcement. Both test programs will utilize full-scale beams to test the performance of the limit state in the presence of ASR with two-directional reinforcement replicating key features of the B Electrical Tunnel.

- The out-of-plane shear testing will test the performance of a reinforced concrete section with selective placement of transverse reinforcement to target a shear failure in a region with only two-directional reinforcement.

- The reinforcement anchorage testing will test the performance of lap splices in flexure without transverse reinforcement.

This method of determining adequacy is essentially proof testing and is deemed a "more precise method" by the ACI code. The full-scale tests will provide a definitive assessment of the nominal margin inherent in the designs for each limit state targeted and any apparent strength reductions due to various degrees of ASR.

A final assessment will need to revisit the current evaluation screening criteria based on the available literature with criteria derived from the full-scale testing programs. The structural components will need to be reevaluated based on the screening criteria derived from the full-scale structural testing. The reevaluation will include the following:

- Structural components that were identified as requiring an evaluation in the initial screening for the interim assessment.
- Structural components that screened out for the interim assessment but screen in based on the future condition.
- Any structural components walked down after the interim assessment that screen in based on the future condition.

7 Evaluation of Structural Attachments

This section assesses the impact of ASR on anchorages for safety-related systems and components. This assessment employs the following approach:

- Identify the types of anchors used in safety-related applications and the related design bases.
- Perform testing to document the impact of ASR-induced cracking on anchor performance. The scope of testing is based on the applicable anchor types, likely limiting failure modes and expected impact of ASR on anchor performance.
- Evaluate the ability of safety-related anchors at Seabrook Station to perform their design basis safety function given the currently documented extent of ASR. This evaluation includes identification of additional work required to complete a final assessment.

This assessment focuses on anchor performance under tensile, rather than shear, loading. This is because the performance of anchors under tensile loading is more directly impacted by concrete properties than under shear loading. Reacting a tensile anchor load requires formation of a series of inclined compressive struts that radiate from the anchor head to the concrete surface. The strut compressive force is maintained by a tension field in the concrete (See Reference 9.2.7, Section 8.3.3). Shear failure, however, is primarily due to shear stress in the anchor shank, accompanied by local crushing of the concrete at the surface. Unless the anchor is located near a free surface, shear failure by concrete breakout is not a possible failure mode.

7.1 Design Description

A variety of anchor designs and configurations are used in safety-related applications at Seabrook Station. Anchors can be divided into two broad categories:

- Cast-in-Place Anchors: These anchors are suspended in the supporting structure's formwork and concrete is then cast around it. Load is transferred through bearing from the anchor directly to the concrete. Cast-in place anchors in use at Seabrook Station include embedded plates (with Nelson studs), embedded uni-strut type channels (with embedment studs), Richmond Stud and anchor bolts.
- Post-Installed Anchors: These anchors are installed by drilling a hole in the existing concrete. The anchor assembly transfers load to the concrete through friction and/or bearing at the anchor/hole interface. Post-installed anchors in use at Seabrook Station include both expansion anchors (e.g. Hilti Kwik Bolts) and undercut anchors (e.g., Drillco Maxi-Bolts).

7.1.1 Anchor Applications

Cast-in-place and post-installed anchors are used primarily for pipe supports, electrical cable supports, and component anchorages. The following describes the types of anchors typically used in each application.

Pipe Supports

Pipe supports are typically anchored using post-installed Hilti Kwik Bolts, although Drillco Maxi-Bolts are used in some applications. In addition, some larger support (e.g., pipe whip restraints) and piping anchor designs use cast-in-place embedded steel plates with Nelson Studs. Review of Seabrook Station design documentation shows that the following anchor types are used in safety-related applications at the plant:

- Hilti Kwik Bolt 1: Standard (Carbon Steel), Super, and Stainless Steel
- Hilti Kwik Bolt 2: Carbon Steel and Stainless Steel
- Hilti Kwik Bolt 3
- Drillco Maxi-Bolt
- Embedded Plates with Nelson Studs

The Seabrook Station Pipe Support Qualification Standard (Reference 9.6.4) identifies key documents used in the design of the pipe support anchorages at the plant. The UE&C Pipe Support Design Guideline Documents (References 9.6.5 and 9.6.6) provide the basis for support design at Seabrook Station from original construction through today. Note that Kwik Bolt 2 and Kwik Bolt 3 bolts were approved for use after initial construction as a replacement for Kwik Bolt 1 bolts upon discontinuation of the Kwik Bolt 1 product line.

Electrical Supports

Electrical and I&C cabling and component supports are typically anchored using post-installed Hilti Kwik Bolts, cast-in-place plates or cast-in-place Unistrut-type embedded channels (with embedment studs). The scope of anchors used in safety-related electrical applications is based on review of References 9.4.1, 9.4.2, 9.4.3 and 9.4.4.

Design guidance for electrical supports at Seabrook Station is provided in the UE&C Technical Guide for the Design and Analysis of the Electrical Conduit System (Reference 9.6.7). Note that, as it relates to anchor design (e.g., anchor bolt allowable loads, applied factor of safety), the design guidance provided in Reference 9.6.7 is consistent with that for pipe supports in References 9.6.5 and 9.6.6.

Component Anchorages

Anchorages for safety-related components are typically cast-in-place, using embedded steel plates, or ductile steel bolts (e.g., Richmond anchors). Each component anchorage is individually designed and analyzed. As such there is no generic guidance regarding component anchorage sizing or design. However, the potential impact of ASR induced cracking on these anchors will be identical to that of other deeply embedded cast-in-place anchors, such as those used in pipe whip restraints. Therefore, specific embedment plate configurations are not relevant to this assessment.

7.1.2 Range of Anchorage Types in Service

This section documents the range of anchorage types accepted for service at Seabrook Station. Review of sample support calculations and discussion with plant personnel indicates that most, if not all, of the wide range of anchorage types (including sizes and embedment depths) accepted for use are currently installed in safety related applications.

Hilti Kwik Bolts

As discussed above, the Seabrook Station design basis permits the use of Hilti Kwik Bolt 1, 2, 3, and Super (for use in deeper embedment applications). The use of Kwik Bolt 2 and 3 designs was added after plant construction due to discontinuance of the Kwik Bolt 1 line by Hilti. NextEra Energy has performed equivalency evaluations for Kwik Bolt 2 and 3 embedment depths and spacing requirements to ensure that bolts used to replace Kwik Bolt 1 designs satisfy existing design strength requirements.

Seabrook Station design basis documents permit the use of Kwik Bolt (1, 2, 3, and Super) sizes ranging from 0.25 inch to 1.25 inches with minimum embedment depths from 1.125 inches to 13.25 inches, respectively. It should be noted that the Pipe Support Design Guides (References 9.6.5 and 9.6.6), which provide minimum embedment depths and allowable loads, provide recommended embedment depths for design purposes that are deeper than the minimum. For example, a 0.625 inch Kwik Bolt 1 has a minimum embedment depth of 2.75 inches, with a recommended embedment depth of 4.5 inches for design purposes. Discussions with plant personnel indicate that the "design" embedment depths are used whenever possible, although the minimum embedment depths are used when deeper embedments are not practical.

Drillco Maxi-Bolts

Drillco Maxi-Bolts were permitted for use in pipe supports at Seabrook Station (References 9.6.5 and 9.6.6), although their use has been discontinued and is not permitted in new support designs (Reference 9.6.4). When permitted, Maxi-Bolts ranging from 0.5 inch to 1.25 inches with minimum embedment depths from 6 inches to 12.5 inches, respectively, were authorized for use.

Cast in Place Anchors

A wide range of cast in place anchor types are in use at Seabrook Station. The plant employs a variety of embedded plates and channels anchored with headed studs from several manufacturers (e.g., Nelson studs, embedded Unistruts, Richmond anchors and inserts and anchor bolts). All of these anchor types are deeply embedded (typically >6 inches), and designed such that the limiting failure mechanism is yielding of the ductile steel insert, rather than through failure of the surrounding concrete. Based on these design similarities, the potential impact of ASR on their performance is expected to be consistent between designs.

7.1.3 Relevant Concrete Design Information

The compressive strength and depth of reinforcing steel ("cover depth") are both relevant to the assessment of anchor performance.

Safety-related structures at Seabrook Station are typically constructed with concrete with a minimum 28 day compressive strength (f'_c) of 3,000 psi. Analysis of core samples taken during original construction (Reference 9.2.11) shows that the actual average compressive strength of 3,000 psi concrete was 4,359 psi with all individual test results at least 3,500 psi.

Reinforcing Steel

The concrete structures at Seabrook Station contain two directional reinforcing steel. Note 23 of Reference 9.4.5 indicates that typical cover depth (the depth of reinforcing steel) beneath the concrete surface of safety-related structures at Seabrook Station is 2 inches, with the exception of the external wall surfaces, which have a cover depth of 3 inches. Based on this, the typical cover depth in regions of interest for this evaluation (i.e., areas with embedded supports also exhibiting ASR cracking) is 2 inches. The reinforcing steel is typically in a grid configuration spaced at 12 inches.

7.2 Failure Mechanisms and Design Philosophy

This section discusses anchor failure mechanisms and the design philosophy typically used in anchor design to ensure reliable operation. This information provides the basis for anchor testing performed as part of this effort to determine the impact of ASR-induced cracking on anchor performance at Seabrook Station.

7.2.1 Anchor Bolt Failure Modes

The load path due to tensile loading of concrete-embedded anchors loads the steel fastener itself, the interface between the fastener and concrete, and creates a tension field in the surrounding concrete. Anchor capacity is typically limited by three general failure modes (Reference 9.1.4):

- Tensile Steel Fastener Failure – Tensile loading results in yielding and eventual failure of the steel fastener shank. Commonly applied concrete anchor design philosophy is to embed the anchor sufficiently deep such that tensile failure of the steel fastener is the limiting failure mode. As discussed above, this practice appears to have been employed at Seabrook Station in the design of cast-in-place anchors. However, practical limitations associated with the installation of post-installed anchors often prevent the use of this approach. As such, many post-installed anchors in service at Seabrook Station appear to be limited by other failure modes.

- Pullout/Pull-Through – Pullout occurs when the anchor pulls completely out of the hole, usually accompanied by local crushing of the concrete above the anchor head. Note that partial pullout of the anchor, followed by failure due to concrete breakout at the shallower embedment depth is not uncommon. Pull-through is a similar failure mode, occurring when the anchor shank separates from the expansion clip or sleeve. Note that this failure mode is only applicable to expansion anchors, such as Kwik Bolts.

- Concrete Breakout – Failure due to propagation of a roughly conical fracture surface in the concrete, extending from the tip of the anchor to the concrete surface. The angle of the fracture surface (relative to the surface plane) increases from 35° at shallow embedments to 45° at deeper embedments.

7.2.2 Anchor Design Philosophy

Typical anchor design practices encourage anchor designs to have a ductile failure mode, which is consistent with the strength design philosophy of reinforced concrete in flexure. The anchor failure mechanism is controlled by requiring yielding of the steel anchor prior to brittle concrete failure. This design practice permits redistribution of the load to adjacent anchors, providing greater design margin. Review of relevant design guidance shows that design practices at Seabrook Station are largely consistent with this philosophy. Most anchorages used at Seabrook Station (including all cast-in-place anchors) are designed such that brittle concrete breakout failure is not the limiting failure mechanism. However, review of design drawings and discussions with plant personnel indicate that, in cases where post-installed anchors were used in low-load applications (e.g., electrical conduit supports), smaller expansion anchors were embedded to depths at which the limiting failure mechanism would likely be concrete breakout.

7.3 DESIGN BASIS

The design of safety related concrete structures at Seabrook Station is governed by ACI 318-71 (Reference 9.1.1) which requires that anchorages must be capable of developing adequate reinforcement strength without damage to the concrete and that their adequacy be demonstrated with testing (Reference 9.1.1, Section 12.12). In addition, NextEra Energy has committed to the requirements of IEB 79-02 (Reference 9.7.3) for post-installed anchor design. In accordance with this commitment, a safety factor of 4 on mean failure load is used for the design of pipe supports with post-installed anchors. Note that this safety factor is applied to all safety-related post-installed anchors at Seabrook Station. Review of relevant design documentation indicates that design practices at Seabrook Station are consistent with these requirements.

Post-installed anchor allowable loads are based on the following:

- Hilti Kwik Bolts: The allowable loads for all Kwik Bolts specified for use at Seabrook Station are based on qualification testing performed by Hilti or a third party (Abbot Hanks). The tensile load capacities were determined by unconfined tensile testing in unreinforced test specimens (none of the qualification test reports reviewed noted the presence of reinforcing steel). Allowable loads are based on the tested mean failure load with an applied safety factor of four. Note that the qualification test values are based on an actual compressive strength (f'_c) of 3,000 psi. Hilti Kwik Bolt design loads used at Seabrook Station are taken from the following documents:
    - Hilti Kwik Bolt 1: Abbot A. Hanks Test Report 8783 R (FP 44412 – Reference 9.6.8)
    - Hilti Kwik Bolt Super: Abbot A. Hanks Test Report 8786 (FP 44412 – Reference 9.6.8)
    - Hilti Kwik Bolt 3: Hilti Product Technical Guide Supplement (FP 100174 – Reference 9.6.9)

- Hilti Kwik Bolt 2: DRR 92-64 (Reference 9.2.10). The design loads provided in DRR 92-64 are consistent with those specified in the Hilti Kwik Bolt 2 Technical Guide (Reference 9.6.10), with a Safety Factor of 4 applied.

- Drillco Maxi-Bolt: The Station Pipe Support Design Guidelines (Reference 9.6.6) indicates that these anchors are embedded to sufficient depth that the failure is limited by tensile failure of the anchor steel. Review of anchor dimensions and the material specification shows that the design allowable loads are based on tensile failure of the anchor bolt shank with a safety factor of 4 applied. While the basis for specified minimum embedment depths is not provided, scoping calculations indicate that the minimum embedment depths provide 40% margin between shank tensile failure and theoretical concrete breakout failure, based on the 45° shear cone method, a commonly used approach during the Seabrook original construction period.

Cast in place anchors (e.g., Nelson studs or embedded unistrut-type channels) are typically designed with embedment depths such that the limiting failure mode is ductile failure of the anchor steel. Note that in the case of cast in place anchors, the applied safety factors are consistent with vendor recommendation, and are in some cases less than 4.

7.4 Testing on Anchor Performance in ASR-affected Concrete

MPR sponsored testing at FSEL at The University of Texas at Austin to determine the impact of ASR-induced cracking on anchor performance. The testing was supervised by Dr. Richard Klingner, an expert in concrete anchor design and performance who has authored several technical reports for the NRC providing guidance on the assessment of anchor performance in the nuclear industry (e.g., NUREG/CR-5434 and NUREG/CR-5563). Reference 9.2.6 documents the test program and includes the FSEL test report (Reference 9.2.7) as an appendix. The test program is summarized below.

7.4.1 Description

The objective of the testing was to better understand the performance of post-installed anchors (both expansion and undercut) under tension when subjected to a range of ASR-induced cracking. Both the pullout/pull-through and concrete breakout failure mechanisms were investigated. Ductile steel failure, the third anchor failure mechanism, is not affected by changes in concrete characteristics.

- Pullout/Pull-Through – Pullout/pull-through capacity is derived from friction at the concrete-anchor interface in expansion anchors (e.g., Hilti Kwik Bolts). Confined tensile testing was performed to determine if ASR degradation resulted in local changes in the concrete properties that reduced the friction at the anchor-concrete interface.

- Concrete Breakout – Concrete breakout capacity is impacted by cracking, which interferes with the tension field formed at the concrete surface to resist the compression field formed by tensile anchor loading. It is expected that ASR-induced cracking will impact anchor behavior similar to cracking due to other mechanisms, which is a well understood phenomenon. Unconfined testing was performed to assess the impact of ASR-induced cracking on concrete breakout capacity.

Test Specimens

The tests completed to date were performed on an existing box girder at FSEL. The lateral faces of the tested girder where the pullout tests were performed have vertical reinforcement at a depth of two inches with a spacing of five inches; there was no horizontal reinforcement along the side faces of the girder. The specified compressive strength of the concrete is 9.5 ksi.

The girder exhibits varying levels of ASR-induced cracking, ranging from levels consistent with the worst ASR cracking observed at Seabrook Station to cracking much more severe than at Seabrook Station. The cracking severity was quantified using the Combined Cracking Index (CCI) method used during the site walkdowns performed as part of this assessment and mapped to determine appropriate test locations. The horizontal and vertical Cracking Indices were averaged to obtain the Combined Cracking Index. The CCI was devised to have a single parameter to characterize the extent of cracking when there are significant differences between horizontal and vertical CIs due to single direction reinforcement. This approach yields a conservative result when applying test results to anchors at Seabrook Station.

Control tests were performed on an existing test specimen at FSEL unaffected by ASR. The unaffected specimen is similar to the test specimen, with 6-inch reinforcement spacing (in one direction) and a specified compressive strength of 10 ksi (12 ksi tested).

Note that the next phase of the Anchor Test Program includes testing in new test specimens that more closely match the Seabrook Station concrete strength and reinforcing steel configuration. Although the concrete mix design for these specimens will produce a similar compressive strength as the concrete at Seabrook Station, the concrete mix will be specifically designed to produce significant ASR in just a few months. Anchor testing will be performed at different times to capture different levels of ASR degradation.

Confined Tension Tests

Pullout behavior was investigated using confined tension tests in which the anchor was extracted using a center hole ram placed directly against the concrete surface. This method places the concrete surface in compression, preventing failure due to concrete breakout, and ensuring that anchor failure is due to pullout/pull-through. Tests were performed on 5/8-inch Hilti Kwik Bolt 3 anchors, embedded to a depth of four inches. This depth was chosen to be representative of a typical embedment depth used at Seabrook Station and is shallow enough to ensure that ductile failure of the anchor shank is precluded. In addition, tests were conducted with anchors installed with the manufacturer-recommended torque and with reduced torque (approximately ½ manufacturer recommended). The reduced torque tests were performed to assess the impact of potential in-service loss of preload due to concrete relaxation or ASR.

Thirty six confined tests were conducted:

- Ten control tests, performed in new concrete with no ASR; five with full torque and five with reduced torque.

- Sixteen full torque tests, conducted in ASR affected concrete with average Combined Cracking Indices ranging from 0.0 to 18.7 mm/m (significantly beyond the maximum currently observed level of cracking at Seabrook; which is approximately 2.5 mm/m)

- Fifteen reduced torque tests, conducted in ASR affected concrete with Combined Cracking Indices ranging from 1.4 to 4.6 mm/m.

Unconfined Tension Tests

Concrete breakout behavior was investigated using unconfined tension tests, in which the anchor was extracted using a center-hole ram held away from the surface of the concrete by a test fixture. Tests were performed on 5/8-inch Hilti Kwik Bolt 3 and Drillco Maxi-Bolts embedded to a depth of four inches. The depth was chosen to be representative of a typical embedment depth used at Seabrook Station and is shallow enough to ensure that ductile failure of the anchor shank is precluded. Note that the control (non ASR affected test specimen) tests for the Maxi-Bolts were performed at an embedment depth of 3 inches to ensure the failure mode would be concrete breakout. As test results are normalized against theoretical capacity (a function of compressive strength and embedment depth), this has no effect on the test results.

Nineteen unconfined tests were conducted, as listed below.

- Eight control tests, performed in new concrete with no ASR; five with Maxi-Bolts and three with Kwik Bolt 3.

- Nine Maxi-Bolt tests in ASR affected concrete with Combined Cracking Indices ranging from 2.6 to 10.8 mm/m.

- Two Kwik-Bolt tests in ASR affected concrete.

The scope of the unconfined tests was not as large as that for the confined tests due to issues with the quality of the concrete in the girder that limited the portion of the girder that was suitable for testing.

7.4.2 Results

For each test, the anchor load-displacement behavior was recorded and the peak load taken as the failure load. Complete test results are provided in Reference 9.2.6.

Data Normalization

To account for variations in concrete strength and embedment depth, tensile capacities are normalized by the best available theoretical prediction of capacity. This is the tensile breakout capacity predicted by the Concrete Capacity (CC) method, used in current industry design codes, and accepted by the NRC (NUREG/CR-5563 – Reference 9.1.4).

$$N_b = \frac{\psi_c k \sqrt{f_c'} h_{ef}^{1.5}}{F_m}$$

where:

N_b	=	Concrete breakout capacity for a single anchor remote from edges (lb_f)
ψ_c	=	1.0 for cracked concrete: Note that current design codes also provide an un-cracked concrete factor (1.4 for expansion anchors, 1.25 for cast-in-place) which was not used in this evaluation
k	=	17 for expansion anchors. Note that this factor, used in design codes, is conservatively based on 5% fractile, rather than mean failure. To predict mean failure, the k factor is adjusted by dividing by F_m=0.7. This ratio represents standard industry practice, and is based on typical sample sizes and coefficients of variation for breakout test.
f'_c	=	Specified 28-day concrete compressive strength (psi)
h_{ef}	=	Effective embedment depth (in)
F_m	=	0.7; factor to correct from 5% fractile to mean failure. This ratio represents standard industry practice, and is based on typical sample sizes and coefficients of variation for breakout test.

Normalized results for the Hilti Kwik Bolt 3 and Drillco Maxi-Bolt are provided in Figure 7-1 and Figure 7-2, respectively. These figures are taken from Reference 9.2.7.

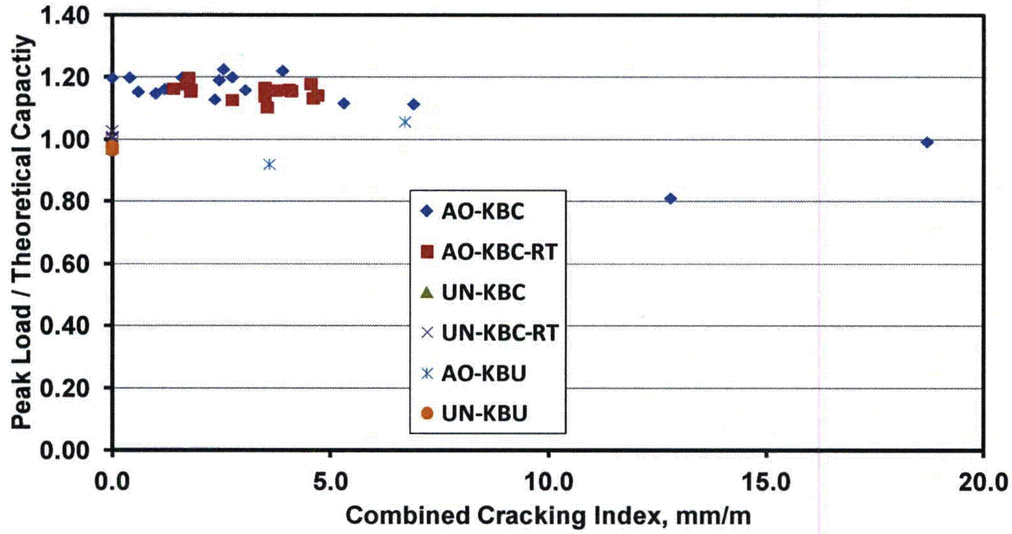

Figure 7-1. Kwik Bolt 3 Tensile Capacity Results

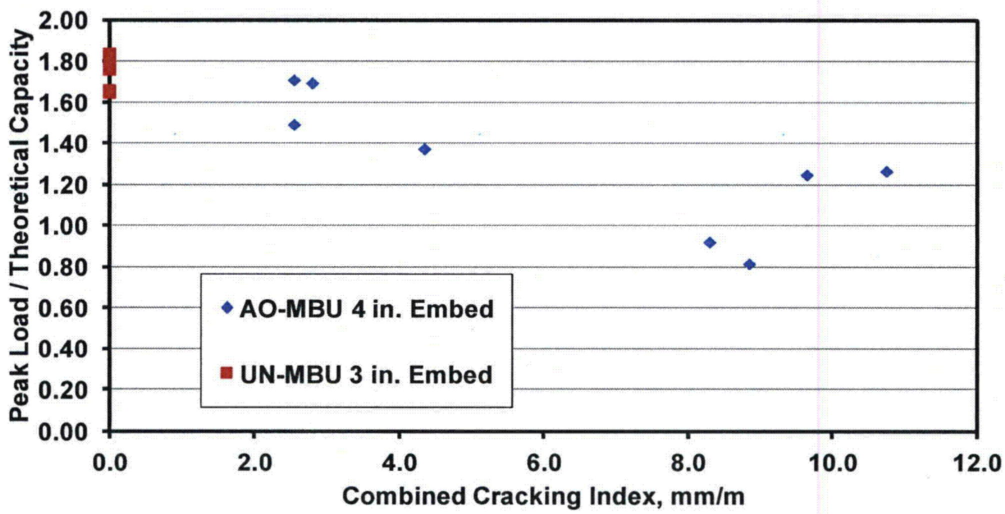

Figure 7-2. Maxi-Bolt Tensile Capacity Results

Legend Notes:

- The first term (AO or UN) identifies a control (UN) or ASR (AO) test.
- The second term (KBU, KBC, or MBU) identifies the bolt and test type. Kwik Bolt (KB) or Maxi-Bolt (MB); confined (C) or unconfined (U) test.
- RT identifies a reduced torque test.

7.4.3 Observations

Review of the test data provided in Figure 7-1 and Figure 7-2 provides the following conclusions/observations.

Hilti Kwik Bolts

- Confined and unconfined tests at low Combined Cracking Indices are consistent with the theoretical capacity calculated using the Concrete Capacity (CC) method.

- Confined test results show essentially no loss of pullout/pull-through capacity at Combined Cracking Indices observed at Seabrook Station (typically less than 2 mm/m, with a maximum of 2.5 mm/m).

- Unconfined tests are inconclusive due to limited data, although the available data show that the values are close to the theoretical capacity calculated using the CC method.

Drillco Maxi-Bolts

- Tests at low Combined Cracking Indices show that the actual capacity significantly exceeds the theoretical capacity calculated using the CC method. This is consistent with a broad body of test data, which has demonstrated that Maxi-Bolt performance is more consistent with that of cast-in-place anchors. (This is discussed in Reference 9.2.7 as well as NUREG/CR-5563; Reference 9.1.4.)

- There is a steady decrease in breakout capacity as the Combined Cracking Index increases. At Combined Cracking Indices typically observed at Seabrook Station, this loss of capacity is approximately 12% (CCI=2 mm/m) relative to the control specimen capacity. The loss of capacity (relative to the control specimen) at the highest observed Combined Cracking Index of 2.5 mm/m is approximately 16%.

7.5 CONCLUSION

Review of the Anchor Test Program scope and results confirms that they are adequately representative of anchors in service at Seabrook Station and consistent with the current Seabrook Station design basis. The Hilti Kwik Bolt 3 design is sufficiently similar to previous Kwik Bolt designs in service at Seabrook Station that it is reasonable to use the Kwik Bolt 3 results when assessing the performance of other Kwik Bolt designs. The Drillco Maxi-Bolt design tested is in service at Seabrook Station; additionally it was selected because its behavior is known to be similar to cast-in-place anchors, such as Nelson studs or Richmond inserts. Review of the test data in Figure 7-1 and Figure 7-2 shows that the capacity of tested anchors in control or low ASR specimens was largely consistent with theoretical capacities. While the Seabrook Station anchor design is based on qualification testing and not capacity calculations, comparison of qualification test results to the nominal (i.e., low ASR impact) capacities shows that the nominal (low ASR impact) test results are consistent with values used for design at Seabrook Station.

The FSEL test report (Reference 9.2.7) concludes that anchor capacity decreases slowly as ASR-induced cracking increases and that this is consistent with the impact of anchor behavior due to any type of cracking. The behavior of anchors in cracked concrete is well understood and is accurately predicted with the CC method. A more detailed discussion is provided in Reference 9.2.7.

Impact of ASR Induced Cracking on Anchor Operability

The test data obtained by FSEL shows that, over the range of Combined Cracking Indices currently observed at Seabrook Station, there is essentially no reduction in pullout/pull-through capacity, and a relatively small (approximately 12%) decrease in concrete breakout capacity. The measured decrease in breakout capacity appears to be predictable and consistent with the loss of capacity due to general concrete cracking. The behavior of anchors in cracked concrete is well understood (accurately predicted with the CC method) and accounted for in concrete design codes. It is concluded that the range of ASR-induced cracking currently observed at Seabrook Station does not adversely impact the operability of safety related concrete anchors in service at the plant. This is supported by several significant conservatisms in the Seabrook Station design basis.

- Design basis allowable loads are based on anchor qualification tests performed in concrete corresponding to a measured compressive strength of 3,000 psi. While the minimum specified 28-day compressive strength at Seabrook Station is 3,000 psi, the average tested strength is 4,359 psi. As concrete breakout capacity is proportional to the compressive strength squared, the relative increase in average anchor capacity is $(4{,}359/3{,}000)^{1/2}=1.21$ (i.e., a 21% increase).

- Design loads are determined using a Safety Factor of 4; however, a Safety Factor of 2 is more appropriate when assessing the operability of concrete anchors.

- Many of the anchors in service at Seabrook Station, with the exception of shallowly embedded Kwik Bolts, have been designed such that concrete breakout is not the limiting failure mode, which is consistent with the recommended design philosophy in current nuclear concrete design codes.

  - Qualification test results used as the basis for the Kwik Bolt 3 design loads (References 9.6.9) show that at most embedment depths (except the most shallow), the initial anchor failure mode is pullout/pull-through, likely followed by breakout at a reduced embedment. While the margin between pullout (which is not affected by low ASR crack indices) and breakout can be difficult to quantify, it does represent significant additional conservatism in the design relative to reduced breakout capacity.

  - Review of the Maxi-Bolt design basis shows that original design basis likely included a 40% margin between steel failure and concrete breakout at minimum specified embedment depths.

- As discussed before, the Kwik Bolt design loads were determined by testing samples and applying a Safety Factor of 4 to the mean failure load. These failure loads are provided in Appendix B normalized to the theoretical capacity with the same approach used in normalizing the FSEL test results. The plots of normalized mean failure load versus embedment depth show a substantial decrease in tested capacity relative to theoretical as embedment depth decreases, particularly for the Kwik Bolt 1 and Kwik Bolt 2 designs. In the worst case, the tested capacity is approximately 25% of that predicted using the CC method and applying the factor for cracked concrete. This is likely due to systematic errors common in older anchor test practices (excerpted from Reference 9.2.7, Section 8.3.1):

- Testing laboratories often did not correctly account for the effects of overall flexure on their unconfined test specimens. As a result, results of unconfined tension tests on deep anchors were conservative.

- Testing laboratories generally did not recognize the difference between cracked and un-cracked concrete. As a result, their tested values were probably representative of what we would get today if we tested a group of anchors installed in concrete with some cracking. Some anchors would coincide with cracks, and others would not.

It appears that the design loads of many Kwik Bolts in service at Seabrook Station are based on tests that significantly underpredicted their capacity in good concrete, such as would be expected to be found at Seabrook Station.

7.6 Future Actions

As discussed above, the anchor testing performed to date has been limited to the use of available existing ASR-affected specimens with a limited number of available test locations. Initial test results have increased our understanding of the impact of ASR-induced cracking on anchor performance and provide confidence in the current operability of the Seabrook Station anchors. Additional testing will be performed to (1) verify applicability of the initial test results to anchor behavior in concrete more representative of that used at Seabrook Station, (2) better understand the potential impact of embedment depth on ASR-induced anchor degradation, (3) quantitatively define the impact of additional ASR-induced cracking on anchor performance relative to the Seabrook Station design basis, and (4) define action levels relative to Cracking Indices to be used in the ASR Aging Management Program.

8 Path Forward

The preceding sections of this report demonstrate that reinforced concrete structures and anchorages at Seabrook Station are acceptable for an interim period. The fact that it took decades to manifest the levels of ASR observed at the plant suggests that the degradation rate is slow. This means that an aging management approach is likely appropriate for ASR degradation of reinforced concrete structures at Seabrook Station. However, a long-term assessment of the impact of ASR must be completed, especially for areas of concern in this interim assessment, before ASR is handled solely as an aging management issue.

The efforts necessary to address the long-term implications of ASR degradation and develop a solid technical basis for the ASR Aging Management Program are outlined below.

8.1 Testing Programs

The testing programs that are underway form the basis for understanding the long-term implications for the structures and for anchors. These programs will support both a long-term structural assessment as well as definition of action levels for an aging management program. These programs are described briefly below. All of the test programs are being conducted at FSEL with the technical and quality assurance oversight by MPR.

8.1.1 Shear Test Program

The Shear Test Program will establish the shear capacity and flexural stiffness of concrete beams without shear reinforcement, which have varying levels of ASR degradation. It will also investigate potential structural modification concepts to restore shear capacity as necessary.

The Shear Test Program will involve testing of large beams designed and fabricated to replicate the limiting wall in the B Electrical Tunnel. The full-scale beam specimens will model realistically the structural details germane to the shear behavior in the walls of the B Electrical Tunnel. The concrete mixture will be as similar as possible to the concrete at Seabrook, with the provision that the mix be adjusted to provide ASR expansion in a reasonable time period. The depth of the beam will be consistent with the wall thickness in the B Electrical Tunnel.

The test program will include a control test and two series of tests with ASR-affected concrete. Each test is described below.

- Control—The control test will provide a baseline by which to judge potential reductions in capacity due to ASR. This test will also be used to quantify the margin available in the structure that is likely above the capacity calculated using ACI 318-71.

- Series 1—The Series 1 tests will quantify the impact of ASR on shear strength and flexural stiffness (EI) for multiple levels of ASR degradation. The lowest level of degradation tested will be similar to that observed in the B Electrical Tunnel; subsequent tests will be performed at higher levels of degradation.

- Series 2—The Series 2 tests will investigate approaches for restoring structural capacity should ASR reduce the structural capacity. The levels of ASR investigated in the Series 2 tests will be based on insights from the Series 1 tests.

ASR expansion of the concrete will be assessed by monitoring surface cracking in the concrete cover, and monitoring the expansion of the "structural core" of the concrete (i.e., the concrete in the middle of the beam that is constrained by the inner and outer rebar mats). Inspection of the surface cracking will include measurement of the Combined Cracking Index to allow comparison to the Combined Cracking Indices determined in ASR walkdowns at Seabrook Station – present and future.

8.1.2 Lap Splice Test Program

The Lap Splice Test Program will establish the performance of reinforcement anchorage and flexural stiffness in concrete beams without transverse reinforcement which have varying levels of ASR degradation. The testing will focus on lap splices, which is the limiting design feature with regard to reinforcement anchorage. The testing will also investigate potential structural modification concepts to compensate for apparent degradation of reinforcement anchorage if necessary.

The Lap Splice Test Program will be very similar to the Shear Test Program. Key differences between the two programs relate to specimen preparation and how the beams are loaded during testing. The specimens for the Lap Splice Test Program will include reinforcement lap splices and will be designed to ensure that reinforcement anchorage will be the limiting failure mode, whereas the specimens for the Shear Test Program will not include lap splices and will be designed to ensure that shear is the limiting failure mode. Conduct of the test including the parameters measured will be nearly identical to the Shear Test Program with the exception of how the beams are loaded. It will include control tests and two series as outlined above for the Shear Test Program.

8.1.3 Anchor Test Program

The Anchor Test Program will establish the performance of expansion anchors and undercut anchors in ASR-affected concrete. The expansion anchors tested will be from the Hilti Kwik Bolt family, which is a common type of anchor used at Seabrook Station. The undercut anchors used will be Drillco Maxi-Bolts, which are used in some applications at Seabrook. These undercut anchor results will provide insights for other types of anchors including embedments and cast in place anchors.

The Anchor Test Program, which started in December 2011, consists of two test series: Girder Test Series and Block Test Series.

- The Girder Test Series uses an existing ASR-affected bridge girder available at FSEL. The objective of this test series was to obtain data necessary to support assessment of anchors for the interim period.

- The Block Test Series will use new blocks that are representative of typical wall configurations at Seabrook Station in terms of concrete strength and reinforcing steel configuration. The objective of this testing is to study in a more systematic fashion the impact of ASR on the capacity of anchors. Testing will be performed at different levels of ASR as the blocks age. The tests will vary embedment depth, installation effort and other parameters.

ASR expansion of the concrete will be assessed by monitoring surface cracking in the concrete cover. Inspection of the surface cracking will include measurement of the Combined Cracking Index to allow comparison to the Combined Cracking Indices determined in ASR walkdowns at Seabrook Station – present and future.

The initial Girder Test Series is complete. The Block Test Series will commence shortly.

8.2 DEGRADATION RATE

The rate 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 degradation rate is to monitor expansion of the concrete *in situ*. The walkdowns conducted by MPR (see Section 3.2) provide a baseline for monitoring expansion of the structures due to ASR. NextEra Energy will reinspect the selected areas at periodic intervals to ascertain the change in the Combined Cracking Index, which relates to the bulk expansion due to ASR. Since the test programs will correlate performance of structures and anchors to Combined Cracking Indices, the rate of change in the Combined Cracking Index provides a means for estimating future condition. Per discussions with experts on ASR, it can take two to three years to obtain a reliable estimate of the rate of expansion.

As discussed in Section 3.1.3, NextEra Energy is initiating residual aggregate reactivity testing program to assess the relative portion of reactive silica in the remaining aggregate. This testing is planned for reclaimed aggregate from cores taken in areas with ASR damage and cores taken in areas without ASR damage (controls). Also new aggregate from the quarry used during construction will be tested. The results for the three types of specimens will be compared to obtain a qualitative assessment of the amount of reactive silica remaining (i.e., the relative extent of reaction). If the specimens made with reclaimed aggregate from areas with ASR expands little compared to the other specimens, then there is limited potential for additional ASR expansion at the plant. However, if the specimens with reclaimed aggregate from areas with ASR expand similar to the other specimens, then additional expansion is likely unless the amount of alkali in the pore solution is limiting.

8.3 LONG-TERM ASSESSMENT OF IMPACT OF ASR

The long-term impacts of ASR on plant structures and concrete anchors will be assessed using the results from the test programs described above, with consideration of the potential for additional ASR expansion in the future. Ultimately, the presence of ASR in concrete structures at Seabrook Station will be reconciled with the plant's design basis calculations.

8.4 Aging Management Program Action Levels

The ASR Aging Management Program will include periodic monitoring of plant structures to trend the progression of ASR degradation and to identify when degradation has reached a level requiring action. Action levels will be derived from the test programs described above. These action levels will be based on the observed Combined Cracking Index as this is the parameter being monitored at the plant and the correlating parameter for the testing. When an action level is reached, NextEra Energy will need to take additional action to ensure the given structure or anchors can satisfy their required design basis loads. The additional action may be review of design calculations to ensure there is sufficient margin to accommodate ASR degradation, or potentially plant modifications. The Shear and Lap Splice Test Programs include test series to investigate and qualify modification concepts.

9 References

9.1 Codes and Standards

9.1.1 ACI 318-71, "Building Code Requirements for Reinforced Concrete," American Concrete Institute, 1971.

9.1.2 ACI Committee 408, "Bond and Development of Straight Reinforcing Bars in Tension," (ACI 408R-03), Farmington Hills: American Concrete Institute, 2003.

9.1.3 ACI 318-11, "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, 2011.

9.1.4 NUREG/CR-5563, "A Technical Basis for Revision to Anchorage Criteria," March 1999.

9.1.5 ACI 221.R, "Report on Alkali-Aggregate Reactivity," American Concrete Institute, 2008.

9.2 Reports

9.2.1 Simpson, Gumpertz, & Heger Report RPT-100502 (-1, -2, -3, and -4), "Petrographic Examination of Concrete Wall Core Samples from B Electrical Tunnel at the Seabrook Nuclear Station, Seabrook, NH," dated August 4, 2010. (Seabrook FP No. 100581)

9.2.2 Miller Engineering & Testing letter for Project No. 10.027.NH, Lab No. L100191, dated June 2, 2010. (Seabrook FP No. 100572)

9.2.3 Simpson, Gumpertz & Heger Report RPT-100502 (-5), "Compression Modulus Testing of Cores," dated September 21, 2010. (Seabrook FP No. 100584)

9.2.4 Wiss, Janney, Elstner Associates, Inc. letter dated November 30, 2011, "Compressive Strength Testing of Concrete Cores; Seabrook Station, NH - Project No. 0326-0058-13; WJE Project No. 2011.4688." (Seabrook FP No. 100661)

9.2.5 Simpson, Gumpertz, & Heger Report RPT-100502.02 (-1, -2, -3, -4, -5, -6, -7, -8, -9, -10), "Petrographic Examination and Analysis of Hardened Concrete," dated July 1, 2011. (Seabrook FP No. 100636)

9.2.6 MPR-3722, "Strength Testing of Anchors in Concrete Affected by Alkali-Silica Reaction," Revision 0. (Seabrook FP No.100718)

9.2.7 Ferguson Structural Engineering Laboratory Test Report, "Performance of Post-Installed Anchors in Existing ASR-Affected Concrete," dated April 12, 2012. (Included as Appendix C in MPR-3722)

9.2.8 Simpson, Gumpertz, & Heger Report RPT-100502-11, "Compression Modulus Testing of Cores," dated July 27, 2011. (Seabrook FP No. 100635)

9.2.9 MPR-3704, "Summary of Alkali-Silica Reaction Walkdown Results," Revision 1, April 2012. (Seabrook FP No. 100705)

9.2.10 DRR 92-64, Hilti Kwik-Bolt II, Rev. 0.

9.2.11 Foreign Print 100348, "Statistical Analysis – Concrete Compression Test Data," Revision 0.

9.3 Calculations

9.3.1 SGH Calculation No. 110594-CA-01, "Field Investigation," Revision 0. (Seabrook FP No. 100700)

9.3.2 SGH Calculation No. 110594-CA-02, "Laboratory Testing and Data Analysis – Determination of Material Properties of ASR-Affected Concrete," Revision 0. (Seabrook FP No. 100696)

9.3.3 SGH Calculation No. 110594-CA-03, "Three Dimensional Dynamic Analysis of Containment Enclosure Building," Revision 0. (Seabrook FP No. 100714)

9.3.4 SGH Calculation No. 110594-CA-04, "Static Finite Element Analyses and Study on ASR Impact on the Containment Enclosure Building," Revision 0. (Seabrook FP No. 100715)

9.4 Drawings

9.4.1 Drawing 1-NHY-300228, "Conduit System - Notes & Typ. Details," Revision 31.

9.4.2 Drawing 1-NHY-300229, "Cable Tray Systems - Notes & Typ. Details," Revision 25.

9.4.3 Drawing 1-NHY-504606, "I&C Instrumentation Supports – Volume 1, "Revision 0.

9.4.4 Drawing 1-NHY-504607, "I&C Instrumentation Supports – Volume 2, "Revision 0

9.4.5 Drawing 9763-F-101842, "Concrete General Notes & Reinforcing Splice Lengths," Revision 14.

9.5 Technical Papers

9.5.1 "Structural Implications of ASR: State of the Art," Bayrak, O., February 2012. (Seabrook FP No. 100697)

9.5.2 "Perspectives on ACI 318-71," Bayrak, O. March 2012. (Seabrook FP No. 100717)

9.5.3 Farny & Kosmatka, "Diagnosis and Control of Alkali-Aggregate Reactions in Concrete," Portland Cement Association, 1997.

9.5.4 "Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction in Transportation Structures," U.S. Dept. of Transportation, Federal Highway Administration, January 2010, Report Number FHWA-HIF-09-004.

9.5.5 "Structural Effects of Alkali-Silica Reaction: Technical Guidance on the Appraisal of Existing Structures," Institution of Structural Engineers, July 1992.

9.5.6 ORNL/NRC/LTR-95/14, "In-Service Inspection Guidelines for Concrete Structures in Nuclear Power Plants," December 1995.

9.5.7 Fournier, B.; Berube, M.A.; Thomas, M.D.A.; Smaoui, N. and Folliard, K.J., "Evaluation and Management of Concrete Structures Affected by Alkali-Silica Reaction: A Review." Ottawa: CANMET Materials Technology Laboratory, Natural Resources Canada, 2004. Report Number MTL 2004-11.

9.5.8 Chana, P.S. "Bond Strength of Reinforcement in Concrete Affected by Alkali-Silica Reaction," Crowthorne: Transport and Road Research Laboratory, Department of Transport, 1989, Contractor Report 141.

9.5.9 Ahmed, T.; Burley, E. and Rigden, S. "The Static and Fatigue Strength of Reinforced Concrete Beams Affected by Alkali-Silica Reaction," ACI Materials Journal Vol. 95 No. 4 (1998): 376-388.

9.5.10 den Uijl, J.A. and Kaptijn, N. "Structural Consequences of ASR: An Example of Shear Capacity," Heron Vol. 47 No. 2 (2002): 125-139.

9.6 Seabrook Specifications, Design Guides, and Manuals

9.6.1 Seabrook Station Updated Final Safety Analysis Report, Revision 14.

9.6.2 Seabrook Station Standard No: 9763-006-14-1, "Specification for Furnishing, Detailing, Fabricating, and Delivering Reinforcing Bars," Revision 10.

9.6.3 Seabrook Station Standard No: 9763-006-69-7, "Specification for Standard Concrete Mixes," Revision 2.

9.6.4 Engineering Design Standard 36260," Pipe Support Qualification," Revision 1.

9.6.5 Foreign Print 18558, "UE&C Pipe Support Design Guidelines," Revision 1 plus changes.

9.6.6 Foreign Print 18559, "UE&C Additional Information for Pipe Support Design Guidelines," Revision 1 plus changes.

9.6.7 "UE&C Technical Guide for the Design & Analysis of the Electrical Conduit System, Revision 1.

9.6.8 Foreign Print 44412, "Hilti Anchor and Fastener Design Manual," Revision 0.

9.6.9 Foreign Print 100174, "Hilti Kwik Bolt 3 Product Technical Guide," Revision 0.

9.6.10 H-437C, "Hilti Fastening Technical Guide, April 1991.

9.7 Other References

9.7.1 MPR Document 0326-0058-06, "Scope for Alkali-Silica Reaction Walkdowns," Revision 1, dated February 3, 2012. (Seabrook FP No. 100642)

9.7.2 MPR Document 0326-0058-02, "Procedure for Alkali-Silica Reaction Walkdowns and Assessment Checklist," Revision 2, dated January 13, 2012. (Seabrook FP No. 100641)

9.7.3 IE Bulletin 79-02, "Pipe Support Base Plate Designs Using Concrete Expansion Anchors," Revision 2.

9.7.4 Winter, N., Understanding Cement: An introduction to cement production, cement hydration and deleterious processes in concrete, 2009.

9.7.5 Hobbs, D., Alkali-silica reaction in concrete, 1988.

9.7.6 R. W. Clough & J. Penzien, "Dynamics of Structures," McGraw Hill, 1975.

9.7.7 AR 581434, "Reduced Concrete Properties Below Grade In 'B' Electrical Tunnel Exterior Wall," Revision 0.

9.7.8 AR 01664399, "Reduced Concrete Modulus Of Elasticity Below Grade In Containment Enclosure Building, RHR Equipment Vaults, EFW Pumphouse, And Diesel Generator Fuel Oil Tank Rooms," Revision 0.

A
Structural Component Calculations

This appendix contains MPR Calculation 0326-0058-63, "Review of Structural Calculations for ASR-Affected Structures," Revision 1.

MPR

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CALCULATION TITLE PAGE

Client: NextEra Energy Seabrook, LLC	Page 1 of 22 plus App. A–L
Project: Seabrook ASR Evaluation	Task No. 0326-1105-0058-03
Title: Review of Structural Calculations for ASR-Affected Structures	Calculation No. 0326-0058-63

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
J. L. Hibbard Sections 1-3, 5 & 6 and App. A & J May 23, 2012	K. Gantz Sections 1-6, App. A, D, E, F, I, J, K, & L May 23, 2012	O. Bayrak Reviewer, May 23, 2012	1
K. Gantz App. B, C, G, & H May 23, 2012	R. Maisel App. B, C, G, & H May 23, 2012	R. B. Keating Reviewer & Approver May 23, 2012	
R. Maisel App. D, E, F, I, & K May 23, 2012			
R. Vayda Section 4 & App. L May 23, 2012			

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	MPR Associates, Inc. 320 King Street Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0326-0058-63	J L Hubbard	Kevin Ys	

Revision	Affected Pages	Description
0	All	Initial Issue
1	1, 2, 6-8, 13-16, 18-20, A-1, A-6, B-1, B-4, C-1, C-2, C-4 through C-6, D-1, D-5, E-1, E-2, E-3, E-7, E-8, F-1, F-2, F-4, G-1, H-1, I-1, I-4, J-1, K-1, L-1 through L-5, all pages are reissued as Rev. 1	The calculation was revised to incorporate the following changes: • Editorial and grammatical changes, where necessary. • Page 16 - Deleted statement that the Containment Enclosure Building has a combined cracking index greater than 1.5 mm/m. • Page 18 - Corrected documented acceptance criteria for lap splice and embedment lengths, differentiating between the size of rebar present. This change did not affect the review procedure or results, and was only for clarification. • Page 19 - Clarified margin acceptance criteria. • Page 20 - Clarified that verification of the completeness of the reviewed calculations in addressing all applicable ACI Code requirements was not performed as part of the review. • Page C-1 - Corrected table references. • Pages C-2, C-4, and C-5 - Updated anticipated margin for the out-of-plane shear evaluation of the West wall of CBST1. • Pages E-2 and F-2 - Corrected the table headings (incorporated pen-and-ink changes from Revision 0 of the calculation). • Page K-1 - Clarified the location of the North wall that was reviewed. • Pages L-3 and L-5 - Clarified the difference between the reduction in capacity and the effect on margin. All of the aforementioned changes did not have any impact on the conclusions presented in the Revision 0 version of the calculation. The only numerical result that changed was the anticipated margin for the out-of-plane shear evaluation of the West wall of CBST1. The anticipated margin increased slightly in this area compared to that reported in the Revision 0 version. All changes are indicated with revision bars.

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 QA Form QA-3.1-2, Rev. 0

Calculation No.	Prepared By	Checked By	Page: 3
0326-0058-63	J L Hubbard	Kevin Getz	Revision: 1

Table of Contents

1.0 PURPOSE

This calculation documents a review of design basis calculations for selected Seabrook buildings that are affected by Alkali Silica Reaction (ASR). The review:

- Documents the margin in the design basis calculation for each evaluation of the building in the ASR-affected areas.
- Identifies the evaluations that are adversely affected by ASR degradation of the concrete. This assessment is based on ASR effects on performance of reinforced concrete in a structure as discussed in Section 4.0.
- Identifies analysis options that could be employed to increase the margin in evaluations that are adversely affected by ASR degradation of the concrete. These options were only identified when the documented margin was less than the anticipated degradation in concrete performance from ASR. The anticipated reduction in concrete performance was based on the criteria for ASR-affected structures from Section 4.0. Scoping calculations were used to estimate the margin that would exist after reanalysis with unnecessary levels of conservatism removed.
- Identifies evaluations that may not have sufficient margin, even with unnecessary analysis conservatisms removed.

Calculations for eleven ASR-affected areas were reviewed as part of this effort. The eleven areas were selected to represent a reasonable sampling of the ASR-affected regions of most concern for plant operability. In addition, the buildings with the worst ASR based on the walkdown assessments were included in the review. Thus, the eleven areas selected for the review are a biased selection that increases the probability of identifying issues related to ASR effects on plant structures. Table 1-1 identifies these building locations and the corresponding design basis calculations that were reviewed. The specific reference and revision of each calculation that was reviewed are identified in Section 6.0.

Table 1-1. Building Locations with ASR in the Review

Structure	Elevation	Room Number	Structural Components of Concern	Reference Calculation
RHR Vault	-61 ft. up to -16 ft.	Various	All Walls	PB-30
Emergency Feedwater Pumphouse	-26 ft. up to grade (20 ft.)	EFST1	North and East Walls	EF-4
Electrical Tunnel 'B'	-20 ft. to 20 ft.	CBST1	North, East, and West Walls and Floor Slab	CD-20, C-S-1-10159
RCA Tunnel	0 ft. & 5 ft.	Unit 1 Tunnels	NE Wall @ El. 0' & All Cored Walls	SG-1, CD-10, WB-69
Diesel Generator Building	-16 ft.	DG102	East Wall	CD-18
Primary Auxiliary Building	-6 ft.	PB205	South Wall, East Wall (South Portion)	PB-20, WB-82
Primary Auxiliary Building Mechanical Penetration	-34 ft.	MF102	North Wall (Column near NE corner of room)	EM-31
Electrical Tunnel 'B'	-20 ft.	EF101	North and South Walls and Floor Slab	EF-4, EF-11
MS/FW Pipe Chase (East)	Above Grade (> 20 ft.)	Exterior	East Wall	EM-19
Cooling Tower	Above Grade (> 20 ft.)	Unit 1 Exterior	South Wall & North Pipe Chase Bump out	CT-53, CT-28
Service Water Pumphouse	Above Grade (> 20 ft.)	Exterior	North Wall of SW Bump out & South Wall	CW-29, SBSAG-1MA

2.0 SUMMARY

Detailed results of the calculation reviews are provided in Appendices A through K. A summary of the review results is provided in Table 2-1.

This review addressed eleven buildings/rooms. Of these, six buildings/rooms were shown to have sufficient margin (either documented in the design basis calculations or after potential removal of unnecessary conservatism included therein) to accommodate potential degradation of the concrete by ASR. These buildings/rooms are identified in Table 2-1. Five buildings/rooms have at least one evaluation for which the margin in the calculation is not sufficient for the potential degradation of the concrete by ASR, even with reanalysis to remove unnecessary levels of conservatism. These buildings/rooms are also identified in Table 2-1. The specific evaluations with insufficient margin to accommodate concrete degradation due to ASR are identified in Table 2-2 through Table 2-6.

These results are based on a review of the design basis calculations and recovery of margin that is available in analysis space. The results do not address margin that can potentially be recovered through other avenues, such as from anticipated full-scale structural testing.

The potential conservatism in the structural calculations is an estimate based on scoping evaluations. MPR provided informal checks of these estimates to assure they were reasonable. These scoping calculations are not included in this calculation and the margin recovery estimates are not QA results. The identification of conservatism is an estimate of the likely margin that could be obtained if the design basis calculation were revised.

Table 2-1. Summary of Review

Structure	Evaluated Locations with ASR	Appendix	Results of Calculation Review
RHR Vault	All Walls -61 ft. to -16 ft.	A	See Table 2-2
Emergency Feedwater Pumphouse, EFST1	North and East Walls, -26 ft. to grade (20 ft.)	B	See Table 2-3
Electrical Tunnel 'B', CBST1	North, East, and West Walls and Floor Slab, -20 ft. to 20 ft.	C	No Evaluations of Concern for ASR
RCA Tunnel, Unit 1 Tunnels	NE Wall @ Elev. 0' & All Cored Walls	D	See Table 2-4
Diesel Generator Building, DG102	East Wall, -16 ft.	E	See Table 2-5
Primary Auxiliary Building, PB205	South Wall, East Wall (South Portion), -6 ft.	F	No Evaluations of Concern for ASR
Primary Auxiliary Building Mechanical Penetration, MF102	North Wall (Column near NE corner of room), -34 ft.	G	No Evaluations of Concern for ASR
Electrical Tunnel 'B', EF101	North and South Walls and Floor Slab[1], -20 ft.	H	No Evaluations of Concern for ASR
MS/FW Pipe Chase (East), Exterior	East Wall, above grade (> 20 ft.)	I	No Evaluations of Concern for ASR
Cooling Tower, Unit 1 Exterior	South Wall & North Pipe Chase Bump out, above grade (> 20 ft.)	J	See Table 2-6
Service Water Pumphouse, Exterior	North Wall of SW Bump out & South Wall, above grade (> 20 ft.)	K	No Evaluations of Concern for ASR

[1] The floor slab for 'B' Electrical Tunnel was not included in this review. The design basis calculation was not available.

Table 2-2. RHR Vault, Various Rooms
Evaluations Without Sufficient Anticipated Margin
ASR Location: All walls, -61 ft. to -16 ft.
Appendix: A

Wall	Evaluation	ASR Effect	Documented Margin (%)	Margin after Recovering Conservatism (%)	Required Margin Considering Potential ASR Degradation (%)
El. -45' 4' ext. wall	Out of Plane Shear	Concrete Shear Capacity Reduced 25%	24%	24%	25%

Table 2-3. Emergency Feedwater Pumphouse, Room EFST1
Evaluations Without Sufficient Anticipated Margin
ASR Location: North and East Walls, Below Grade, Elev. -26 ft. to 20 ft.
Appendix: B

Wall	Evaluation	ASR Effect	Documented Margin (%)	Margin after Recovering Conservatism (%)	Required Margin Considering Potential ASR Degradation (%)
East	Vertical Reinforcement for In-Plane Moment El. 0' to 27'	Embedment & Splice Length Increased 17%	4.4%	6%	17%

Table 2-4. RCA Tunnel
Evaluations Without Sufficient Anticipated Margin
ASR Location: Unit 1 North End of East Wall and Core Bore Locations RCAW 1-4
Appendix: D

Wall	Evaluation	ASR Effect	Documented Margin (%)	Margin after Recovering Conservatism (%)	Required Margin Considering Potential ASR Degradation (%)
NE Corner of Tunnel	Vertical Reinf: Flexure and Compression	Embedment & Splice Length Increased 40%	18%	18%	40%
NE Corner of Tunnel	Horizontal Reinf: Shear	Embedment & Splice Length Increased 40%	39%	39%	40%
West Wall (Control Bldg) - Core Bore RCAW-1&2	Flexure and Tension	Embedment & Splice Length Increased 40%	18%	18%	40%

Table 2-5. Diesel Generator Building, Room DG102
Evaluations Without Sufficient Anticipated Margin
ASR Location: East Wall – Elev. -16 ft.
Appendix: E

Wall	Evaluation	ASR Effect	Documented Margin (%)	Margin after Recovering Conservatism (%)	Required Margin Considering Potential ASR Degradation (%)
East	Flexure	Embedment & Splice Length Increased 17%	5.3%	10%	17%

Table 2-6. Cooling Tower
Evaluations Without Sufficient Anticipated Margin
ASR Location: Unit 1 South Wall and North Pipe Chase Bump Out Exterior Above Grade
Appendix: J

Wall	Evaluation	ASR Effect	Documented Margin (%)	Margin after Recovering Conservatism (%)	Required Margin Considering Potential ASR Degradation (%)
South, El. 32 to 39', Cols. A-D	Horiz. reinf. for in-plane shear, bending, and tension	Embedment & Splice Length Increased 17%	2%	14%	17%
South, El. -8 to 21'	Out of plane shear	Concrete Shear Capacity Reduced 25%	7%	7%	25%
South, El. 21 to 45'	Vert. reinf. for bending	Embedment & Splice Length Increased 17%	-2.5%[2]	-2.5%	17%
South, El. >50', Cols. D-K	Vert. reinf. for bending and tension	Embedment & Splice Length Increased 17%	p. / % 34 / 5.0 35 / 6.3 35 / 8.9 36 / 8.9 37 / 8.9 38 / 11	p. / % 34 / 5.0 35 / 6.3 35 / 8.9 36 / 8.9 37 / 8.9 38 / 11	17%

[2] This result is for the worst case finite element for the wall being evaluated.

3.0 ASSUMPTIONS

3.1 Assumptions that Require Verification

Calculation WB-82 (Reference 1.6) was reviewed to determine the documented margin for the South Wall of Room PB205 of the Primary Auxiliary Building. However, the problem statement for calculation WB-82 states that the calculation applies to Unit 2 only. The applicability of this calculation to Unit 1 must be verified.

3.2 Assumptions with a Basis

1. The screening criterion for lap splices was established based on References 4 and 5, and is based in part on test data from Reference 8. The test data for lap splices discussed in Reference 8 are for No. 5 bar, while the reinforcement bar size at Seabrook is larger, typically No. 8 or larger. In addition, the test protocol in Reference 8 was direct pull out of reinforcement from test blocks, which produces different structural behavior compared to lap splices in a wall (Reference 10). It is assumed that the test data of Reference 8 provides a reasonable estimate of the lap splice performance in the Seabrook buildings, though, based on the differences between the test samples and the Seabrook structures, the test data is expected to be conservative. Accordingly, the screening criterion is viewed as a reasonable best-engineering estimate in the absence of direct data.

2. The screening criterion for out-of-plane shear was established based on References 4 and 5, and is based in part on test data from Reference 9. The test data for shear discussed in Reference 9 is for 5 in. by 3 in. beams. The test specimens are smaller in size than the walls of the Seabrook buildings. It is assumed that the test results of Reference 9 provide a reasonable estimate of the performance of the walls of the Seabrook buildings with transverse shear. This is expected to be a conservative assumption. Accordingly, the screening criterion is viewed as a reasonable best-engineering estimate in the absence of direct data.

4.0 ASR STRUCTURAL CONCERNS AND SCREENING CRITERIA

4.1 ASR Structural Concerns

Reference 4 discusses the structural implications of ASR. Table 4-1 is a summary of the results provided in Table 4 of Reference 4. Table 4-1 lists limit states, e.g., shear and flexure, and the effect of ASR on strength. The last column of Table 4-1 is an assessment of whether the limit state and the potential loss of strength is a concern for the Seabrook Station. The rationale for this judgment is provided in the footnotes to the table.

Table 4-1. Assessment of Limit State Effects on ASR-Affected Structures

Limit State		Lower Bound Effect of ASR	Concern for Seabrook Buildings?
Axial Compression		Moderate loss of strength: up to 18% loss	No[1]
Flexure		No significant loss of strength or stiffness: up to 7% loss	No[2]
One-Way Shear	with transverse reinforcement	No significant loss of strength or stiffness: more than 16% gain	No
	without transverse reinforcement	High variability among similar specimens: up to 25% loss	Yes
Two-Way Shear		No significant loss of strength or stiffness: up to 9% loss	No[2]
Reinforcement Anchorage	with transverse reinforcement	No significant loss of strength or stiffness: up to 10% loss	No[2]
	without transverse reinforcement	Significant loss of strength of 40%[3]	Yes

Notes:

1. Axial compression is a concern for columns in compression or for walls in compression with high compression relative to the applied flexure loads, i.e., on the interaction diagram, failure would occur in the compression-controlled region. This requirement is not applicable to Seabrook structures. This was confirmed by the detailed reviews in Appendices A-K. There are no column compression evaluations in the areas affected by ASR and the flexure evaluations for walls in compression are governed by the tensile limits for the reinforcement, i.e., on the interaction diagram, failure would occur in the tension-controlled region.
2. The lower bound loss of strength is minor. Average loss of strength, as reported in the text of Reference 4, is less than the lower bound loss of strength. For this operability assessment, it is reasonable to use no loss of strength for this limit state.
3. Average loss of strength (Reference 8).

Conclusions from Table 4-1 are:

- ASR has potential to reduce the strength of reinforced concrete at locations of reinforcement lap splices and at locations of reinforcement straight bar embedment (i.e., embedments without hooks) in areas where a three-dimensional reinforcement cage is not provided. Sufficient length is required in the reinforcement lap splice length and in the embedment length to fully develop the strength of the reinforcing steel.

- ASR has the potential to reduce the strength of the concrete to resist out-of-plane shear loads in areas where a three-dimensional reinforcement cage is not provided.

- ASR has no significant effects on flexure, one-way shear with transverse reinforcement, two-way shear, and reinforcement anchorage with transverse reinforcement.

ASR effects on compression are not a consideration for the Seabrook buildings included in this review.

Sections 4.2 and 4.3 below include descriptions for the development of screening criteria for the calculation reviews specific to reinforcement anchorage without transverse reinforcement and one-way shear without transverse reinforcement, respectively. The screening criteria are used in the calculation reviews to sort shear and reinforcement anchorage evaluations into those of potential concern and those that are not a concern. The screening criteria are developed using the strength reductions from Table 4-1 and conservatism in ACI[3] acceptance criteria as discussed in Reference 5.

4.2 Reinforcement Lap Splices and Embedment

Strength reductions of 40% for reinforcement lap splices in ASR-affected concrete are identified in Table 4 of Reference 4 and in Reference 8. This is the average strength reduction, which is appropriate for this operability assessment. This potential reduction is based on reinforcement pullout testing in concrete without transverse reinforcement as described in Reference 8. The potential lap splice strength reduction of 40% is likely conservative because the reinforcement pullout testing specimens in Reference 8 used a low or moderate concrete cover-to-bar-diameter ratio relative to Seabrook, and therefore is expected to provide lower strength results. The experimental study in which 40% anchorage strength reduction was measured employed No.5 bars. These results are applied to the generally larger reinforcement bars at the Seabrook Station (see Assumption 1 in Section 3.2).

Conservatism in the ACI Code equations for reinforcement lap splice strength of 23% is documented in ACI Code Committee reports per Reference 5 for reinforcement size No. 7 and

[3] ACI Code is used throughout this calculation to refer to ACI 318-71 (Reference 3).

greater. This conservatism is applicable for elements with two-dimensional rebar, i.e., no transverse reinforcement. The lower bound of the data is what forms the basis for ACI 318-71 code requirements. For this operability assessment, the use of the average value of 23% is appropriate.

Based on References 4 and 5, a criterion of 17% is justified for reinforcement lap splices for reinforcement size No. 7 and greater.[4] This is the potential reduction in strength of reinforcement lap splices and reinforcement anchorage due to ASR.[5] This is the criterion that was used in this review to screen reinforcement lap splice evaluations into those of potential concern and those that are not of concern.

Lap splice length and embedment length are important to three types of evaluations: (1) reinforcement to carry bending moments, (2) reinforcement to carry in-plane shear loads, and (3) for minimum reinforcement requirements.[6] These are the evaluations that are flagged in this review for further scrutiny.[7]

4.3 Out-of-Plane Shear

Potential strength reductions of up to 25% for out-of-plane shear in ASR-affected concrete are identified in Table 4 of Reference 4. This potential reduction is based on testing of 5" x 3" concrete prisms without transverse reinforcement (Reference 9). The results of the testing had high variability with a maximum enhancement of shear strength of 12% and a maximum reduction of 25%. The potential out-of-plane shear strength reduction of 25% is conservative because it is the maximum reported reduction in the test program. Though the test specimens are smaller in size than the walls at Seabrook, the test results are used to develop screening criteria for the Seabrook buildings (see Assumption 2 in Section 3.2).

Conservatism in the ACI Code equations for shear strength of approximately 50% is documented in ACI Technical Committee reports per Reference 5. This conservatism is applicable for elements with rebar in two directions, i.e., no transverse reinforcement, and with wall thickness 24" or less. The lower bound of the data is what forms the basis for ACI 318-71 code

[4] For reinforcement sizes No. 6 and smaller a 40% reduction in strength is used as the screening criterion.

[5] The criterion of 17% is the arithmetic sum of +23% (ACI Code conservatism) and -40% (ASR reduction in strength). The ACI 318-71 equations are not linear and can require an iterative approach. A scoping analysis determined that using the sum of the two considerations is more conservative than comparing the two considerations through the ACI design equations (Appendix L).

[6] The review assures that lap splices and anchorage for minimum reinforcement are adequately sized, considering possible degradation in strength of lap splices or anchorage from ASR. For walls, the minimum reinforcement is primarily for shrinkage, thermal expansion, and serviceability concerns. If the splices are not appropriately sized to carry these loads, the splices could be compromised and the splices could not then carry design basis loads.

[7] The straight length of rebar hook embedments is not impacted by ASR. The hook ends of the rebar are deeply embedded in the concrete and cannot pull out. Justification for this is provided by Reference 8, which shows little reduction in embedment strength for embedments away from the edge of the concrete.

requirements. For this operability assessment, the use of average values is appropriate. Based on References 4 and 5, the following criteria apply to an out of plane shear assessment:

- For walls 24" thick and less, there is no strength reduction due to ASR.[8]
- For walls greater than 24" thick, there is a 25% reduction in shear capacity due to ASR.

5.0 APPROACH

5.1 Buildings and Locations

Table 1-1 includes a list of the eleven buildings and rooms for which the design basis calculations were reviewed. This list of calculations is derived from References 6 and 7, which provide the basis for selection of these buildings and rooms. Two criteria were used to select the buildings/rooms for the review:

- Buildings and rooms for which there is an existing operability issue were included in the review. There are five buildings/rooms with an existing operability issue: containment enclosure building, RHR equipment vaults, EFW Pumphouse, DG fuel oil tank rooms and the 'B' electrical tunnel (Reference 6). These rooms were included in the review with the exception of the containment enclosure building. The effects of ASR on the containment enclosure building are being evaluated in a separate effort by NextEra Energy. In addition, the RCA tunnel was included in the review based on investigations related to wall cores.
- Buildings and rooms with a combined cracking index greater than or equal to 1.5 mm/m were included in the review. These are the rooms that have the most severe ASR. Reference 7 provides the cracking index for each room in the plant walkdown assessments for ASR. Reference 7 also describes the cracking index and how it was measured. There is one building with a cracking index greater than 1.5 mm/m that was not included in the review. This is the Main Steam and Feedwater East Pipe Chase (MF207), which was not included because the cracking was identified in nonstructural grout and this is not considered to represent the condition of the structural concrete (Reference 7).

In summary, the eleven areas selected for the review comprise a biased selection that increases the probability of identifying issues related to ASR effects on plant structures. This selection of buildings/rooms, however, does not address every building with some degree of ASR and therefore is not a complete review of ASR-affected structures.

[8] This conclusion applies when the evaluation is based on a minimum required compressive strength of 3,000 or 4,000 psi, as appropriate to the building being reviewed. This conclusion does not apply when the value of the compressive strength used in the calculation is based on test data. For these cases, the reduction in shear strength is 25% and no credit is taken for the conservatism identified in Reference 5.

5.2 Scope of Review

An outline of the steps performed for each calculation review is provided below.

- Identify the evaluations in each calculation listed in Table 1-1 that address the building rooms and walls that show evidence of ASR.

- Identify the evaluations in each calculation that address the design of the wall/slab to carry design basis loads or to address minimum required reinforcement. All reinforcement (including minimum required reinforcement) is required by the design basis Code of Record (Reference 3) to be developed by providing appropriate length lap splices and properly anchoring reinforcement at supports. Evaluations unrelated to the ability of the structure to carry design basis loads or assess minimum required reinforcement were not considered. For example, reinforcement placement requirements to control cracking from service moments were not included in the review.

- Document the concrete physical properties used in the calculation (e.g., concrete minimum specified compressive strength, modulus of elasticity, and Poisson's Ratio).

- Document the calculated amount of rebar required for each evaluation and the actual amount of installed rebar as shown on the construction drawings.

- Document the calculated margin in the evaluation. The margin is expressed as the percentage: $Margin = 100 * \left[\frac{Capacity - Demand}{Capacity}\right]$. The capacity or demand may be expressed as a force, moment, stress, or rebar area, dependent on how the information was presented in the calculation.

- Identify the evaluations for which ASR is a concern:

  1. Out of plane shear evaluations for walls greater than 24 in. thick for which the shear margin in the design basis calculation is less than 25%.

  2. Evaluations which rely on reinforcement lap splice strength or embedment strength and which meet each of the following criteria:

     - The evaluation credits reinforcement for flexure (with or without axial compression or tension) or in-plane shear. Minimum required reinforcement evaluations per the limits prescribed in Reference 3 are also considered.

     - The margin for required reinforcement area, $100 * \left[\frac{A_{s(present)} - A_{s(required)}}{A_{s(present)}}\right]$ as defined in the previous bullet, is less than 17% for no. 7 bars and greater, and 40% for no. 6 bars and smaller (A_s is the reinforcement area). Note that minimum reinforcement requirements may be governing if flexure and shear loads are small.

– The wall/slab being evaluated has reinforcement anchored with lap splices or reinforcement straight-bar embedment.

- Identify conservatisms in the original design calculation with the potential to increase the margin in the evaluation or alleviate the ASR-degradation concern (see discussion on documentation in Section 5.3). The potential conservatisms that were considered are:

  – Eliminating overly-conservative simplifying assumptions, e.g., calculating a wall bending moment as a two-way slab rather than a one-way slab where wall aspect ratios permit such analysis.

  – Eliminating unnecessary levels of conservatism in the calculation of the applied loads.

  – Use of a more sophisticated analysis method, e.g., a finite element analysis to more accurately calculate the distribution of load. Simple finite element models were prepared to estimate the potential gain with this approach.

  – Taking credit for the actual amount of reinforcement in the wall/slab if this is greater than the amount of reinforcement required to be in the wall/slab per the calculation. The basis for this is Reference 3, Section 12.5(d).[9]

  – Taking credit for adjacent reinforcing steel lap splices that are staggered rather than aligned. The overlap length for lap splices in Seabrook buildings is calculated to be equal to $1.7l_d$, which is a Class C lap splice (Reference 2 and Reference 3, 7.6.1.3). A Class C lap splice is required when more than one-half the bars are lap spliced within the required lap length, $1.7l_d$ (Reference 3, Section 7.6.3.1.1). Seabrook construction drawings show that some walls have staggered lap splices in which one-half or less of the bars are lap spliced within $1.7l_d$. In this case Reference 3, 7.6.3.1.1 requires only a Class B splice, which has a development length of $1.3l_d$.

[9] Reference 3, Section 12.5(d) permits the development length of reinforcement in a flexural member to be reduced by a factor related to excess reinforcement (A_s required/A_s provided). Reference 3, Section 7.6.3.1 states, "Splices in regions of maximum moment preferably should be avoided. Where such splices must be used, they shall be lapped, welded, or otherwise anchored for their full f_y." There are cases in this calculation in which excess reinforcement has been used to recover margin at a location of high moment. The basis for this is as follows: (1) Section 7.6.3.1 provides a design philosophy to assure that splice length is not the limiting factor at the locations in a structure that are most highly loaded, (2) Section 12.5(d) provides the minimum requirement that must be met to assure design loads can be carried without failure of reinforcement at splices or other locations that require the development of reinforcement, and (3) this calculation is an operability assessment and recovery of margin by a technically reasonable approach is appropriate.

The factor increase in lap splice length over that required by Reference 3 is: $\frac{1.7}{1.3} = 1.31$.

- Taking credit for a reduction in required splice length when the lap splice is in a low-stress area (rebar stress below $0.5f_y$ per Section 7.6.3.2 of Reference 3). If the lap splices are not staggered, splices shall meet the requirements of Class B splices ($1.3l_d$ development), and if no more than three quarters of the bars are lap spliced within a required lap length, the splices shall meet the requirements of Class A splices ($1.0l_d$ development).

- Using alternate applicable equations from the ACI Code.

- Determining if the area of interest is affected by ASR based on the walkdown assessments.

The options considered to improve the calculation margin are generally related to the analysis and the actual construction details. Other potential sources of conservatism such as full-scale testing are outside the scope of this calculation.

- Estimate anticipated margin from the above methods. This is the margin that might be obtained with a reanalysis. The estimation is based on a scoping evaluation and is provided for information (see discussion on documentation in Section 5.3).

- Identify the evaluations for which ASR is a potential operability concern, taking credit for the potential margin increase that could be obtained from a reanalysis. The evaluations that are a concern are those that meet the screening criteria of this section and for which the anticipated margin with reanalysis does not meet the acceptance criteria set forth in Sections 4.2 and 4.3.

5.3 Identification of Analysis Conservatism and Documentation

As discussed in the previous section, for cases where an evaluation did not have sufficient documented margin to eliminate ASR concerns, an estimate was made of the margin that could be recovered from the evaluation by removing unnecessary levels of conservatism. The estimate of the margin that can be recovered is documented in the detailed review table provided in each appendix. Footnotes are provided to describe the methods used to recover margin. There is no further documentation provided of the calculations to recover margin. As such, the margin recovery is an estimate of potential over-conservatism that could be recovered if the design basis calculation were to be revised. The estimate of margin is not a documented result and is not a QA result. MPR provided informal checks of these estimates to assure they were reasonable.

5.4 Items Not Included in the Review

Items that were not included in the review are as follows:

- The review did not include an objective of identifying potential errors in the design basis calculations. Nevertheless, some errors were identified in the course of the review and these were brought to the attention of NextEra Energy in separate correspondence. Those errors are not identified or addressed in this calculation.
- The review did not assess whether all building walls and features were analyzed or whether all evaluations required by the ACI Code were performed, i.e., the review did not address the comprehensiveness of the design basis calculations.

6.0 REFERENCES

1. NextEra Energy Seabrook Calculations:

    1.1. Calculation No. PB-30, "PB RHR Vault (030)," Revision 9.

    1.2. Calculation No. EF-4, "EMG Feedwater House & Electrical Penetration Area," Revision 9.

    1.3. Calculation No. CD-20, "Control and Diesel Generator Building (050), Design of Mats at El. -20'-0" and El. 0'-0" and Walls Below Grade for Electrical Tunnels (Control Building)," Revision 4.

    1.4. Calculation No. SG-1, "Non-Essential Switchgear Room (170), Design of Non-Essential Switch Gear Room," Revision 2.

    1.5. Calculation No. CD-10, "Control and Diesel Generator Building (050), Design of Substructure for RCA Walkway Under Control Building," Revision 1.

    1.6. Calculation No. WB-82, "Waste Processing (Including Tank Farm Area) 080, Design of Concrete Walls Below El. 25'-0"," Revision 1.

    1.7. Calculation No. CD-18, "Control and Diesel Generator Building (050), Design of Mats and Walls Below Grade for Fuel Oil Tank Area," Revision 5.

    1.8. Calculation No. PB-20, "Primary Auxiliary Building (030), Concrete – West Wall (Col. Line A)," Revision 4.

    1.9. Calculation No. EM-31, "Tunnels- Pipe, Electric & Passage (150), Concrete Design – Mech. Penetration Area," Revision 6.

    1.10. Calculation No. WB-69, "Tank Farm - Unit 1, Design of Walls & Slabs for Pipe Tunnel between Column Lines 0.5 & 2.3," Revision 7.

    1.11. Calculation No. EF-11, "EMG Feedwater House & Electrical Penetration Area (160), Verification of Reinforcement in South Wall – Unit 1," Revision 1.

    1.12. Calculation No. EM-19, "MS & FW Pipe Chase – East," Revision 7.

    1.13. Calculation No. CT-53, "Service Water Cooling Towers (140), Design of South Wall at Line 4 to El. 46'-0"," Revision 1.

    1.14. Calculation No. CT-28, "Design of South Wall from El. 46'-0" to 77'-6", North Wall Similar," Revision 6.

1.15. Calculation No. CW-29, "Service and Circulating Water Pumphouse, Design of Walls Above Grade, Roof Slab, & Hatch Covers," Revision 7.

1.16. Calculation No. C-S-1-10159, "'B' Electrical Tunnel Transverse Shear Evaluation Supplement to Calculation CD-20," Revision 0.

1.17. Calculation No. SBSAG-1MA, "Category I Structures, Tornado Missile and Light Aircraft Impact Protection," Revision 1.

2. NextEra Energy Seabrook Drawing No. 9763-F-101842, "Concrete General Notes & Reinforcing Splice Lengths," Revision 14.

3. ACI 318-71, American Concrete Institute Building Code Requirements for Reinforced Concrete.

4. O. Bayrak, "Structural Implications of ASR, State of the Art," February 2, 2012.

5. O. Bayrak, "Perspectives On ACI 318-71 Shear Strength And Lap Splice Performance," March 30, 2012.

6. NextEra Energy Seabrook Prompt Operability Determinations (POD):

   6.1. AR 581434, "Reduced Concrete Properties Below Grade In 'B' Electrical Tunnel Exterior Wall," Revision 0.

   6.2. AR 01664399, "Reduced Concrete Modulus Of Elasticity Below Grade In Containment Enclosure Building, RHR Equipment Vaults, EFW Pumphouse, And Diesel Generator Fuel Oil Tank Rooms," Revision 0.

7. MPR Report No. 3704, "Seabrook Station: Summary of Alkali-Silica Reaction Walkdown Results," Revision 1.

8. Chana, P.S. Bond Strength of Reinforcement in Concrete Affected by Alkali-Silica Reaction. Crowthorne: Transport and Road Research Laboratory, Department of Transport, 1989. Contractor Report 141.

9. Ahmed, T.; Burley, E. and Rigden, S. "The Static and Fatigue Strength of Reinforced Concrete Beams Affected by Akali-Silica Reaction." ACI Materials Journal Vol. 95 No. 4 (1998): 376-388.

10. ACI Committee 408. Bond and Development of Straight Reinforcing Bars in Tension (ACI 408R-03). Farmington Hills: American Concrete Institute, 2003.

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A
RHR Vault, Various Rooms

A.1 ASR Affected Areas

The areas affected by ASR in the RHR Vault are the walls from Elevation -61 ft. to -16 ft.

A.2 Reviewed Calculations

Calculation PB-30 was reviewed. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table A-3.

A.3 Calculation General Methodology

A 2D finite element model of a horizontal slice through the RHR Vault building was used to calculate the distribution of loads on the walls. Properties for the beam elements and the loads in the model were varied to represent the building at different elevations. Hand calculations with one-way slabs to represent walls were used to calculate stress. Separate calculations were used to assess horizontal and vertical reinforcement.

A.4 Evaluations without Sufficient Anticipated Margin

The evaluation for concrete out of plane shear in the external wall at Elevation -45 ft. did not meet the screening criterion of 25% margin. The margin calculated in PB-30 is 24%. This margin was confirmed with a scoping finite element evaluation of the building for the major loads (not including the seismic load, which is small). A summary of the result that did not meet the screening criterion is provided in Table A-1.

Table A-1. Evaluation Without Sufficient Anticipated Margin
RHR Vault, Various Rooms

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
El. -45' 4' ext. wall	Out of plane shear	Concrete Shear Capacity Reduced 25%	24%	No	24%

A.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table A-2 satisfied the screening criteria described in Section 4.0. In some cases, the margin documented in the calculation did not meet the screening criteria, so additional margin was extracted from the evaluation by one of the methods described in Section 5.2. The additional margin was sufficient to satisfy the screening criteria. The applicable evaluations for which additional margin was calculated are indicated in the following table and Section A.6 describes the methods used to extract the additional margin.

For the cases in which the documented margin met the screening criteria, the "Anticipated Margin" column reports the documented margin.

Table A-2. Evaluations With Sufficient Anticipated Margin
RHR Vault, Various Rooms

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
El. -61' 4' ext. wall	Out of plane shear	Concrete Shear Capacity Reduced 25%	12%	Yes	25%
El. -61' 2.5' int. wall	Out of plane shear	Concrete Shear Capacity Reduced 25%	-1.8%	Yes	67%
El. -45' 4' ext. wall	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	15%	Yes	65%
El. -45' 4' ext. wall	Bending	Embedment & Splice Length Increased 17%	37%	No	37%
El. -32' 4' ext. wall	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	14%	Yes	57%
El. -32' 4' ext. wall	Bending	Embedment & Splice Length Increased 17%	35%	No	35%
El. -32' 4' ext. wall	Out of plane shear	Concrete Shear Capacity Reduced 25%	38%	No	38%

Table A-2. Evaluations With Sufficient Anticipated Margin
RHR Vault, Various Rooms

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
El. -32' & -4' 2.5' ext. wall N.	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	11%	Yes	66%
El. -32' & -4' 2.5' ext. wall N.	Bending	Embedment & Splice Length Increased 17%	31%	No	31%
El. -32' & -4' 2.5' ext. wall N.	Out of plane shear	Concrete Shear Capacity Reduced 25%	25%	No	25%
El. -32' 2.5' int. wall	Min. Reinf.	Embedment & Splice Length Increased 17%	23%	No	23%
El. -32' 2.5' int. wall	Bending	Embedment & Splice Length Increased 17%	42%	No	42%
El. -45' & 0' 2.5' ext. wall W.	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	10%	Yes	57%
El. -45' & 0' 2.5' ext. wall W.	Bending	Embedment & Splice Length Increased 17%	32%	No	32%
El. -45' & 0' 2.5' ext. wall N.	Out of plane shear	Concrete Shear Capacity Reduced 25%	51%	No	51%
El. -61' to -45' 4' ext. wall	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	15%	Yes	57%
El. -61' to -45' 4' ext. wall	Bending Reinf. Horiz.	Embedment & Splice Length Increased 17%	39%	No	39%
El. -61' to -45' 2.5' int. walls	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	14%	Yes	57%
El. >-41' 2' ext. wall E.	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	10%	Yes	66%

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Table A-2. Evaluations With Sufficient Anticipated Margin
RHR Vault, Various Rooms

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
El. >-26' 3' ext. wall S.	Min. Reinf. Horiz.	Embedment & Splice Length Increased 17%	19%	Yes	65%
El. >6' 2.5' ext. wall W.	Bending Reinf. Horiz.	NA (No ASR observed in this portion of the wall)	15%	No	15%
El. 6' to 20' 2.5' ext. wall W.	Out of plane shear	NA (No ASR observed in this portion of the wall)	2.7%	No	2.7%
El. >6' ext. wall N.	Min. Reinf. Horiz.	NA (No ASR observed in this portion of the wall)	14%	No	14%
El. -61' 4' ext. wall N.	Bending Vertical Reinf.	Embedment & Splice Length Increased 17%	58%	No	58%
El. -26' 2.5' ext. wall N.	Bending Vertical Reinf.	Embedment & Splice Length Increased 17%	48%	No	48%
El. -61' 4' ext. wall S.	Vert. Reinf. Bending at Col. Line 1	Embedment & Splice Length Increased 17%	33%	No	33%
El. -61' 4' ext. wall N.	In plane shear	NA	65%	No	65%
El. -61' 2.5' int. EW wall	In plane shear	NA	30%	No	30%
El. -61' 2.5' int. NS wall	In plane shear	NA	-1.8%	No	-1.8%

A.6 Methods Employed to Gain Additional Margin

The methods used to gain additional margin in the evaluations were as follows:

- In evaluations for two walls, the reinforcement credited in the calculation was less than the amount of steel that was actually installed. The margin was recalculated using the actual reinforcement present in the walls.
- For eight of the evaluations, the minimum required reinforcement was calculated with the applicable requirement for minimum horizontal reinforcement in a wall from Reference 3, Section 14.2(f). The evaluation in PB-30 conservatively used the minimum reinforcement requirement from Reference 3, Equation 10-1, which is applicable to beams.
- For three walls, the shear stress in the wall was calculated with a finite element model of the RHR Vault. The load applied to the vault was the hydrostatic load. The seismic load, which is small in comparison to the hydrostatic load, was not applied for this scoping evaluation.

Calculation No.	Prepared By	Checked By	Page: A-6
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Table A-3. Review Results
RHR Vault, Various Rooms

Location: RHR Vault
Areas Affected by ASR: All walls, -61' to -16'
Calculations: PB-30, Rev. 9
Drawings: 101510, 101517, 101518, 101519, 101534

As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E (psi)	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
PB-30	El. -61' 4' ext. wall	Out of plane shear	70	Note A	Hydro	3000	3.12E6	NA	115 psi	101 psi	12%	No				Concrete Out of Plane Shear Capacity	Yes		4	25%	No
PB-30	El. -61' 2.5' int. wall	Out of plane shear	70	Note A	Internal flood, seismic	3000	3.12E6	NA	110 psi	112 psi	-1.8%, Note 1	No				Concrete Out of Plane Shear Capacity	Yes		5	67%	No
PB-30	El. -45' 4' ext. wall	Min. Reinf. Horiz.	71	Hand	NA	NA	NA	NA	2.08 in^2/ft, E.F.	1.76 in^2/ft, E.F.	15%	Yes	1.76 in^2/ft E.F.	15%		Splice Lap Length	Yes	2.08 in^2/ft, E.F. Dwg. 101517	2	65%	No
PB-30	El. -45' 4' ext. wall	Bending	72	Note A	Hydro, seismic	3000	3.12E6	NA	393 ft-kip	248 ft-kip	37%	Yes	1.32 in^2/ft, E.F.	37%	End of beam	Splice Lap Length	No	2.08 in^2/ft, E.F. Dwg. 101517			No
PB-30	El. -45' 4' ext. wall	Out of plane shear	72	Note A	Hydro, seismic	3000	3.12E6	NA	109 psi	83 psi	24%	No				Concrete Out of Plane Shear Capacity	Yes		7	24%	Yes
PB-30	El. -32' 4' ext. wall	Min. Reinf. Horiz.	73	Hand	NA	NA	NA	NA	1.69 in^2/ft, E.F.	1.45 in^2/ft, E.F.	14%	Yes	1.45 in^2/ft, E.F.	14%		Splice Lap Length	Yes	1.69 in^2/ft, E.F. Dwg. 101517	2	57%	No
PB-30	El. -32' 4' ext. wall	Bending	74	Note A	Hydro, seismic	3000	3.12E6	NA	322 ft-kip	210 ft-kip	35%	Yes	1.10 in^2/ft, E.F.	35%	End of beam	Splice Lap Length	No	1.69 in^2/ft, E.F. Dwg. 101517			No
PB-30	El. -32' 4' ext. wall	Out of plane shear	74	Note A	Hydro, seismic	3000	3.12E6	NA	109 psi	68 psi	38%	No				Concrete Out of Plane Shear Capacity	No				No
PB-30	El. -32' & -4' 2.5' ext. wall N.	Min. Reinf. Horiz.	76	Hand	NA	NA	NA	NA	1.33 in^2/ft, E.F.	1.19 in^2/ft, E.F.	11%	Yes	1.19 in^2/ft, E.F.	11%		Splice Lap Length	Yes	1.33 in^2/ft, E.F. Dwg. 101518, 101534	2	66%	No

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As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E (psi)	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
PB-30	El. -32' & -4' 2.5' ext. wall N	Bending	76	Note A	Hydro, seismic, mass	3000	3.12E6	NA	148 ft-kip	102 ft-kip	31%	Yes	0.92 in^2/ft, E.F.	31%	End of beam	Splice Lap Length	No	1.33 in^2/ft, E.F. Dwg. 101518, 101534			No
PB-30	El. -32' & -4' 2.5' ext. wall N.	Out of plane shear	77	Note A	Hydro, seismic	3000	3.12E6	NA	110 psi	82 psi	25%	No				Concrete Out of Plane Shear Capacity	No				No
PB-30	El. -32' 2.5' int. wall	Min. Reinf.	Note D	Hand	NA	NA	NA	NA	1.33 in^2/ft, E.F.	1.03 in^2/ft, E.F.	23%	Yes	1.03 in^2/ft, E.F.	23%		Splice Lap Length	No	1.33 in^2/ft, E.F. Dwg. 101534, 101517			No
PB-30	El. -32' 2.5' int. wall	Bending	79	Note A	Hydro, seismic	3000	3.12E6	NA	148 ft-kip	86 ft-kip	42%	Yes	0.77 in^2/ft, E.F.	42%	End of beam	Splice Lap Length	No	1.33 in^2/ft, E.F. Dwg. 101534, 101517			No
PB-30	El. -45' & 0' 2.5' ext. wall W	Min. Reinf. Horiz.	80	Hand	NA	NA	NA	NA	1.05 in^2/ft, E.F.	0.94 in^2/ft, E.F.	10%	Yes	0.94 in^2/ft, E.F.	10%		Splice Lap Length	Yes	1.05 in^2/ft, E.F. PB-30 Note B	2	57%	No
PB-30	El. -45' & 0' 2.5' ext. wall W.	Bending	81	Note A	Hydro, seismic	3000	3.12E6	NA	118 ft-kip	80 ft-kip	32%	Yes	0.71 in^2/ft, E.F.	32%	End of beam	Splice Lap Length	No	1.05 in^2/ft PB-30 Note B			No
PB-30	El. -45' & 0' 2.5' ext. wall N.	Out of plane shear	81	Note A	Hydro, seismic	3000	3.12E6	NA	109 psi	53 psi	51%	No				Concrete Out of Plane Shear Capacity	No				No
PB-30	El. -61' to -45' 4' ext. wall	Min. Reinf. Horiz.	82	Hand	NA	NA	NA	NA	1.69 in^2/ft, E.F.	1.44 in^2/ft, E.F.	15%	Yes	1.44 in^2/ft, E.F	15%		Splice Lap Length	Yes	1.69 in^2/ft, E.F. Dwg. 101534, 101517	2	57%	No
PB-30	El. -61' to -45' 4' ext. wall	Bending Reinf Horiz.	82	Note A, Two-way slab table, Hand	Hydro, seismic	3000	3.12E6	NA	321 ft-kip Note G	196 ft-kip	39%	Yes	1.03 in^2/ft, E.F.	39%		Splice Lap Length	No	1.69 in^2/ft, E.F. Dwg. 101534, 101517			No
PB-30	El. -61' to -45' 2.5' int. walls	Min. Reinf. Horiz.	82	Hand	NA	NA	NA	NA	1.05 in^2/ft, E.F.	0.90 in^2/ft, E.F.	14%	Yes	0.90 in^2/ft, E.F.	14%		Splice Lap Length	Yes	1.05 in^2/ft, E.F. Dwg. 101534, 101517	2	57%	No
PB-30	El. >-41' 2' ext. wall E.	Min. Reinf. Horiz.	83	Hand	NA	NA	NA	NA	0.80 in^2/ft, E.F.	0.72 in^2/ft, E.F.	10%	Yes	0.72 in^2/ft, E.F.	10%		Splice Lap Length	Yes	1.05 in^2/ft, E.F. Dwg. 101534	1, 2	66%	No
PB-30	El. >-26' 3' ext. wall S.	Min. Reinf. Horiz.	83	Hand	NA	NA	NA	NA	1.33 in^2/ft, E.F.	1.08 in^2/ft, E.F.	19%	Yes	1.08 in^2/ft, E.F.	19%		Splice Lap Length	No	1.56 in^2/ft, E.F. Dwg. 101517	1, 2	65%	No
PB-30	El. >6' 2.5' ext. wall W.	Bending Reinf. Horiz.	86	Hand	Hydro, seismic, soil	3000	NA	NA	185 ft-kip Note G	159 ft-kip	14%	Yes	1.44 in^2/ft, E.F.	15%	End of beam	Splice Lap Length	Yes	1.69 in^2/ft, E.F. Dwg. 101534	3		No

MPR QA Form: QA-3.1-3, Rev. 0

As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E (psi)	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
PB-30	El. 6' to 20' 2.5' ext. wall W.	Out of plane shear	87	Hand calc	Hydro, seismic, soil	3000	NA	NA	110 psi	107 psi	2.7%	No				Concrete Out of Plane Shear Capacity	Yes		3		No
PB-30	El. >6' ext. wall N.	Min Reinf Horiz.	87	Hand	NA	NA	NA	NA	1 05 in^2/ft, E.F.	0.90 in^2/ft, E.F.	14%	Yes	0.90 in^2/ft, E.F.	14%		Splice Lap Length	Yes	1.05 in^2/ft E.F. Dwg. 101534	3		No
PB-30	El. -61' 4' ext. wall N.	Bending Vertical Reinf.	95	Interaction Diagram	Hydro, seismic	3000	3.12E6	NA	330 ft-kip	139 ft-kip Note E	58%	Yes	No Calc			Splice Lap Length	No	0.79 in^2/ft, E.F. 101517			No
PB-30	El. -26' 2.5' ext. wall N.	Bending Vertical Reinf.	99	Interaction Diagram	Hydro, seismic	3000	3.12E6	NA	130 ft-kip	68 ft-kip Note F	48%	Yes	No Calc			Splice Lap Length	No	0.79 in^2/ft, E.F. 101517			No
PB-30	El. -61' 4' ext. wall S.	Vert. Reinf. Bending at Col. Line 1	101	Note J, Hand	Not in calc	3000	--	NA	1.58 in^2/ft, Outside Face	1.06 in^2/ft, Outside Face	33%	Yes	1.06 in^2/ft, Outside Face	33%	End of beam	Splice Lap Length	No	1.58 in^2/ft, Outside Face, DWG 101517, Note C			No
PB-30	El. -61' 4' ext. wall N.	In plane shear	103	Hand	Hydro, seismic	3000	NA	NA	110 psi	39 psi	65%	No				NA	No				No
PB-30	El. -61' 2.5' int. EW wall	In plane shear	104	Hand	Hydro, seismic	3000	NA	NA	110 psi	77 psi	30%	No				NA	No				No
PB-30	El. -61' 2.5' int. NS wall	In plane shear	104	Hand	Hydro, seismic	3000	NA	NA	110 psi	112 psi	-1.8%, Note 1	No				NA	No				No

Options to Increase Margin:

1. The actual reinforcing steel area installed is greater than that used in the calculation.
2. The minimum reinforcement in PB-30 was calculated with the lesser of (1) ρ=0.0033 and (2) 4/3 times the steel required for flexure loads based on Reference 3, 10.5.1.1. This reinforcement limit is applicable to beams, but is conservative for walls. The minimum required reinforcement is reduced using the minimum required reinforcement for a wall from Reference 3, Section 14.2(f).
3. The elevation for the evaluation is above the locations that have ASR based on the walkdown assessments (Elevation -61' to -16').
4. Anticipated margin based on a scoping calculation with a 3D finite element model of the RHR vault. The result is for external hydrostatic pressure only. The calculated shear stress is 52 ksi. Accounting for the load factor of 1.4 and the phi factor of 0.85 gives v_u = 86 psi.
5. Anticipated margin based on a scoping calculation with a 3D finite element model of the RHR vault. The result is for internal hydrostatic pressure from an internal flood only. The calculated shear stress is 22 ksi. Accounting for the load factor of 1.4 and the phi factor of 0.85 gives v_u = 36 psi.
6. Not used.
7. A scoping 3D finite element model of the RHR vault confirmed the results reported in calculation PB-30.

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Specific Notes:

A. A two-dimensional beam structure with 17 elements representing the walls of the vault structure was used to solve for the loads and moments due to applied loads with a computer program. A beam model was used with different beam properties to solve for loads and moments at different elevations in the building.

B. PB-30 evaluates #8 on 9 in. centers. Review of Dwg. 101534 shows that the reinforcement is greater than this on the West wall. The actual rebar is not shown because there is uncertainty about the extent of walls covered by the calc.

C. PB-30 indicates the analysis is for vertical rebar (p. 100). The figure showing the #8 on 6 in. centers on p. 102 shows this rebar at the outside face of the base mat extending up into the wall. The figure in the calc is consistent with the actual rebar installed based on Dwgs. 101518 and 101534.

D. The minimum required reinforcement calculation is not in the calculation. The result that is provided in this table was calculated based on rho = 0.0033.

E. There is an error in the moment sign on p. 92. The moment with corrected sign is larger, but margin is estimated to be acceptable and greater than the ASR criterion.

F. There is an error in the moment sign, but the error results in a conservative moment.

G. This quantity was calculated by hand by MPR, and was not in PB-30.

H. Deleted.

I. The negative margin for concrete shear stress was deemed acceptable in the calculation and no rebar was credited with carrying the shear loads. There is an anticipated 25% increase in the concrete shear capacity compared to the values used in the calculation. With this additional capacity, the evaluation will have positive margin without requiring shear reinforcement.

J. Loads were derived in separate calculations, then evaluated in PB-30.

B

Emergency Feedwater Pumphouse, Room EFST1

B.1 ASR Affected Areas

The North and East walls in the Emergency Feedwater Pumphouse Stairwell, room EFST1 show evidence of cracking, potentially by ASR. The areas evaluated for potential degradation by ASR are the North and East walls in EFST1, below grade elevation (El. 20').

B.2 Reviewed Calculations

Calculation EF-4, Revision 9 was reviewed to determine the documented margin for the North and East walls of EFST1. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table B-3.

B.3 Calculation General Methodology

Calculation EF-4, Revision 9 evaluated the walls and a column in the Emergency Feedwater Pumphouse and Electrical Penetration Area. The seismic loads on the structure were calculated using a computer model. Remaining loads were calculated by hand. Only the in-plane structural walls were credited with resisting a given direction of seismic load. In-plane moments were calculated from the shear distribution, treating the walls as cantilevered beams. External walls were additionally designed for resisting out-of-plane moments and shears. These loads were calculated by hand. Once the loads were calculated (either by hand or from a computer model), the walls were evaluated by hand to ACI Code limits.

B.4 Evaluations without Sufficient Anticipated Margin

The evaluation for vertical reinforcement required for the in-plane moment from elevations 0' to 27' in Table B-1 did not meet the screening criteria for lap splices described in Section 4.0. Additional margin was calculated above what was documented in the calculation, but the anticipated margin still did not meet the screening criteria of Section 4.0. Section B.6 describes the method used to extract the additional margin.

More margin may be found from re-evaluating the structure with a detailed computer model, but this effort was beyond the scope of this review and the potential benefit from such efforts is unknown.

Table B-1. Evaluation Without Sufficient Anticipated Margin
Emergency Feedwater Pumphouse, Room EFST1

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
East	Vertical Reinforcement for In-Plane Moment El. 0' to 27'	Embedment & Splice Length Increased 17%	4.4%	Yes	6%

B.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table B-2 satisfied the screening criteria described in Section 4.0. In some cases, the margin documented in the calculation did not meet the screening criteria, so additional margin was extracted from the evaluation by one of the methods described in Section 5.2. The additional margin was sufficient to satisfy the screening criteria. The applicable evaluations for which additional margin was calculated are indicated in the following table and Section B.6 describes the methods used to extract the additional margin. For the cases in which the documented margin met the screening criteria, the "Anticipated Margin" column reports the documented margin.

Table B-2. Evaluations With Sufficient Anticipated Margin
Emergency Feedwater Pumphouse, Room EFST1

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
East	Horizontal Flexure and In Plane Shear EL -26' to 0'	Embedment & Splice Length Increased 17%	3.5%	Yes	25%
East	Out of Plane Shear EL -26' to 0'	Concrete Shear Capacity Reduced 25%	40%	No	40%
East	Vertical Axial Tension EL -26' to 0'	Embedment & Splice Length Increased 17%	37%	No	37%

Table B-2. Evaluations With Sufficient Anticipated Margin
Emergency Feedwater Pumphouse, Room EFST1

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
East	Vertical Axial Load and Flexure EL 0' to 27'	Embedment & Splice Length Increased 17%	89%	No	89%
East	Out of Plane Shear EL 0' to 27'	Concrete Shear Capacity Reduced 25%	65%	No	65%
East	In Plane Shear El. 0' to 27'	Embedment & Splice Length Increased 17%	10%	Yes	32%
North	Vertical Axial Load and Flexure EL -26' to 0'	N/A (Has through-thickness rebar)	1.8%	No	1.8%
North	Out of Plane Shear EL -26' to 0'	N/A (Has through-thickness rebar)	3.2%	No	3.2%
North	Vertical Axial Load and Flexure EL 0' to 27'	Embedment & Splice Length Increased 17%	21%	No	21%
North	Vertical Reinf. for In Plane Moment EL -26' to 0'	Embedment & Splice Length Increased 17%	54%	No	54%
North	In Plane Shear EL -26' to 0'	Embedment & Splice Length Increased 17%	57%	No	57%
North	In Plane Shear EL 0' to 27'	Embedment & Splice Length Increased 17%	10%	Yes	32%

B.6 Methods Employed to Gain Additional Margin

To gain additional margin for the single evaluation that did not meet the screening criteria (see Table B-1), the loads were slightly reduced by using the values documented in Revision 2 of the calculation. The more accurate loads from Revision 2 of the calculation were bounded by the loads from Revision 1, so the new loads were not explicitly evaluated in Revision 2. Accounting for the new loads yielded very little added margin, and the anticipated margin did not meet the screening criteria from Section 4.0.

For the evaluation of the East wall for horizontal flexure, the wall was originally evaluated as a horizontal cantilevered beam, which is appropriate for the part of the east wall nearest to the containment building. Since the stairwell being evaluated is far away from the containment building and has perpendicular walls on the north and south end of the stairwell, the wall is more accurately represented by a pinned-pinned or fixed-fixed beam. The anticipated moment was calculated assuming pinned-pinned boundary conditions, which are more conservative than fixed-fixed boundary conditions.

For the remaining evaluations that required additional margin, the reinforcement credited in the calculation was less than the amount of steel that was actually installed. The margin was recalculated using the actual reinforcement present in the walls.

Table B-3. Review Results
Emergency Feedwater Pumphouse, Room EFST1

Location: Emergency Feedwater Pumphouse Stairwell - EFST1
Areas Affected by ASR: North and East Walls, Below Grade (El. (-)26' to 20')
Calculations: EF-4, Rev. 9
Drawings: 101610, 101612, 101662, 101664, 101665

	As Documented																Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
EF-4	East	Horizontal Flexure and In Plane Shear EL -26' to 0'	27, 29	Hand Calc, One-way slab, Cantilever	Dead, Live, Hydro, Lat. Earth, Dyn. Earth, Seismic	3000	C	C	$A_s = 3.39$ in^2 / ft	$A_s = 3.27$ in^2 / ft	3.5%	Yes	$A_r = 3.27$ in^2 / ft	3.5%	Edge of Wall	Embedment Length/ Splice Length	Yes	$A_s = 3.39$ in^2 / ft	1	25%	No
EF-4	East	Out of Plane Shear EL -26' to 0'	27A	Hand Calc, One-way slab, Cantilever	Hydro + Seismic	3000	N/A	N/A	$V_c = 36.9$ kips	$V_u = 22.1$ kips	40%	No	$A_s = 0$ in^2		N/A	Concrete Out of Plane Shear Capacity	No		N/A		No
EF-4	East	Vertical Axial Tension EL -26' to 0'	28	Computer Run, Reinforcement Table	Dead, Live, Hydro, Lat. Earth, Dyn. Earth, Seismic	3000	C	C	$A_s = 224$ in^2, Notes B & S	$A_s = 141$ in^2	37%	Yes	$A_r = 141$ in^2	37%	El. -26'	Embedment Length/ Splice Length	No	$A_s = 224$ in^2, Notes B & S	N/A		No
EF-4	East	Vertical Axial Load and Flexure EL 0' to 27'	29A-B	Computer Run, Hand Calc	Note E	3000	C	C	$P_c = 3543$ kips	$P_u = 1065$ kips, Note M	70%	Yes	Approx 0.18 in^2 / ft E. F. Note G	89%	El. 0'	Embedment Length/ Splice Length	No	$A_s = 1.58$ in^2 / ft, E. F	N/A		No
EF-4	East	Out of Plane Shear EL 0' to 27'	29B	Computer Run, Hand Calc	Note E	3000	C	C	$V_c = 110$ psi	$V_u = 39$ psi	65%	No	$A_s = 0$ in^2		N/A	Concrete Out of Plane Shear Capacity	No		N/A		No
EF-4	East	Vertical Reinf. for In Plane Moment El. 0' to 27'	29C	Computer Run, Hand Calc	Dead, Live, Hydro, Lat. Earth, Dyn. Earth, Seismic	3000	C	C	$A_s = 3.16$ in^2 / ft	$A_s = 3.02$ in^2 / ft, Note S	4.4%	Yes	$A_s = 3.02$ in^2 / ft, Note S	4.4%	N/A	Embedment Length/ Splice Length	Yes	$A_s = 3.16$ in^2 / ft	5	6%	Yes

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Calc	As Documented														Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
	Wall	Evaluation	Pages	Method	Loads	f'c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
EF-4	East	In Plane Shear El. 0' to 27'	29C	Computer Run, Hand Calc	Dead, Live, Hydro, Lat. Earth, Dyn. Earth, Seismic	3000	C	C	$A_s = 1.2$ in^2 / ft, K	$A_s = 1.08$ in^2 / ft, Notes J & K	10%	Yes	$A_s = 1.08$ in^2 / ft, Notes J & K	10%	N/A	Embedment Length / Splice Length for Horizontal Rebar	Yes	$A_s = 1.58$ in^2 / ft	6	32%	No
EF-4	North	Vertical Axial Load and Flexure EL -26' to 0'	55	Computer Run, Hand Calc	Note E	3000	C	C	$P_c = 675$ kip	$P_u = 663$ kip, Note L	1.8%	Yes	N/A		EL -28'	Embedment Length / Splice Length	No, Note T	$A_s = 2.54$ in^2 / ft, E. F.			No
EF-4	North	Out of Plane Shear EL -26' to 0'	56-57	Computer Run, Hand Calc	Note E	3000	C	C	$A_s = 0.31$ in^2 / 2ft	$A_s = 0.3$ in^2 / 2 ft, Note N	3.2%	Yes	$A_s = 0.3$ in^2 / 2 ft, Note N	3.2%	N/A	N/A	No, Note U	$A_s = 0.31$ in^2 / 2ft			No
EF-4	North	Vertical Axial Load and Flexure EL 0' to 27'	58-59	Computer Run, Hand Calc	Note E	3000	C	C	$P_c = 1404$ kips	$P_u = 1138$ kips, Note L	19%	Yes	$A_s = 0.79$ in^2 / ft E. F., Notes F & G	21%	El. 0'	Embedment Length / Splice Length	No	$A_s = 1.0$ in^2 / ft E. F.			No
EF-4	North	Vertical Reinf. for In Plane Moment EL -26' to 0'	62	Computer Run, Hand Calc	Hydro + Seismic	3000	C	C	$A_s = 5.08$ in^2 /ft, E. F.	$A_s = 2.33$ in^2 / ft, E.F. Note S	54%	Yes	$A_s = 2.33$ in^2 / ft, E. F., Note S	54%	N/A	Embedment Length/ Splice Length	No	$A_s = 5.08$ in^2 /ft, E. F.			No
EF-4	North	In Plane Shear EL -26' to 0'	63	Computer Run, Hand Calc	Hydro + Seismic	3000	C	C	$A_s = 2.54$ in^2 / ft	$A_s = 1.08$ in^2 / ft, Note O	57%	Yes	$A_s = 1.08$ in^2 / ft, Note O	57%	N/A	Embedment Length / Splice Length for Horizontal Rebar	No	$A_s = 2.54$ in^2 / ft			No
EF-4	North	In Plane Shear EL 0' to 27'	63	Computer Run, Hand Calc	Hydro + Seismic	3000	C	C	$A_s = 1.2$ in^2 / ft	$A_s = 1.08$ in^2 / ft	10%	Yes	$A_s = 1.08$ in^2 / ft	10%	N/A	Embedment Length / Splice Length for Horizontal Rebar	Yes	$A_s = 1.58$ in^2 / ft, Note P	6	32%	No

Options to Increase Margin:

1. Evaluate as pinned-pinned or fixed-fixed. Calculation EF-4 evaluated the wall as a cantilever based on the portion of the wall nearest the containment enclosure building, but ASR is not observed in this area. ASR has been observed in the stairwell, and the stairwell east wall is braced by the north and south walls of the stairwell. For pinned-pinned, moment is $wL^2/8$ rather than $wL^2/2$, so moment is reduced by 75%. The flexure load is 29% of the load, so the demand decreases by 22%.
2. Not used.
3. Not used.
4. Not used.
5. Consider reduced load from Rev. 2 of calculation.
6. Account for additional steel present over that which was credited in the calculation.

Specific Notes:

A. Not Used
B. The total available reinforcement was not explicitly calculated in EF-4, though a methodology is prescribed and the reinforcement present is judged to be sufficient. The available reinforcement presented in the table above is calculated based on the prescribed methodology in EF-4. Rebar area presented is for 40% of the wall plus 24 ft of the north wall (considering compression rebar only).
C. The concrete material elastic modulus and Poisson's ratio used in the computer analysis were not provided. These properties were not used in the hand calculation portion of the calculation.
D. Not used.
E. The applied loads and load cases used in the computer run used to get the moment, axial load, and shear on the wall were not identified.
F. Not used.
G. The required rebar was calculated following the same methodology presented in the EF-4 calculation, but with progressively lower values of steel area until the limit was reached.
H. Not used.
I. Not used.
J. The minimum reinforcement area is more limiting and is the required rebar area listed.
K. Rebar area presented is the total from both faces.
L. Load P_u is given for a 25' wall used in a computer model.
M. Load P_u is given for a 22.5' wall used in a computer model.
N. The requirement for minimum shear reinforcement area is bounding.
O. The minimum reinforcement requirement is most limiting.
P. Calculation was performed assuming #7 @ 12" E.F., but the actual design has #8 @ 12" E.F.
Q. Not used.
R. Not used.
S. A technical issue with the derivation of this rebar area was identified during review and has been communicated to the plant by separate correspondence. The technical issue is addressed outside of this review. The margin presented is based on the methodology documented in EF-4.
T. There is through-thickness (3-way) rebar in the lower region of the wall where the moment is highest and where rebar splices are present. When through-thickness rebar is present, there is no significant negative effect on lap splices from ASR.
U. There is through-thickness rebar, so there is no anticipated reduction in shear reinforcement capacity from ASR.

C
Electrical Tunnel 'B', Room CBST1

C.1 ASR Affected Areas

Pattern cracking has been observed in the North, East, and West walls below grade in the Electrical Tunnel B stairwell, room CBST1. As such, the three walls are evaluated for potential degradation from ASR in addition to the floor slab.

C.2 Reviewed Calculations

Calculations CD-20, Revision 4 and C-S-1-10159, Revision 0 were reviewed. Calculation C-S-1-10159, Revision 0 addresses a reduction in concrete compressive strength based on core bore testing. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table C-2.

C.3 Calculation General Methodology

The loads on the East wall of room CBST1 are calculated using a computer model. These loads are then evaluated using hand calculations and an interaction diagram to ensure that the wall can carry the required loads. For all other walls of CBST1 and the slab, the loads on the structure are calculated by hand. The calculated loads are from hydrostatic pressure, seismic loads, and deadweight. Hand calculations and design aids are used to evaluate the areas for the required loads.

C.4 Evaluations without Sufficient Anticipated Margin

None.

C.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table C-1 satisfied the screening criteria described in Section 4.0. In some cases, the margin documented in the calculation did not meet the screening criteria, so additional margin was extracted from the evaluation by one of the methods described in Section 5.2. The additional margin was sufficient to satisfy the screening criteria. The applicable evaluations for which additional margin was calculated are indicated in Table C-1 and Section C.6 describes the methods used to extract the additional margin. For the cases in which the documented margin met the screening criteria, the "Anticipated Margin" column reports the documented margin.

Table C-1. Evaluations With Sufficient Anticipated Margin
Electrical Tunnel 'B', Room CBST1

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
East	Horizontal Minimum Reinforcement	Embedment & Splice Length Increased 17%	8.9%	Yes	54%
East	Vertical Reinforcement: Flexure	Embedment & Splice Length Increased 17%	19%	No	19%
East	In Plane Shear (El. -20' to 0')	Embedment & Splice Length Increased 17%	16.8%	Yes	At Least 17%
East	In-Plane Shear (El. 0' to 21.5')	Embedment & Splice Length Increased 17%, but 5.1% margin required to meet screening criteria (see Section C.6)	11.9%	No	11.9%
West	Horizontal Reinforcement: Flexure	Embedment & Splice Length Increased 17%	26%	No	26%
West	Vertical Reinforcement: Flexure	Embedment & Splice Length Increased 17%	15%	Yes	19%
North	Vertical Reinforcement: Flexure	Embedment & Splice Length Increased 17%	60%	No	60%
West	Out-of-plane Shear	Concrete Shear Capacity Reduced 25% (for f'_c = 4790 psi)	6.2% (for f'_c = 4790 psi)	Yes	3.7% (for f'_c = 3000 psi)
West	Out-of-plane shear	Concrete Shear Capacity Reduced 25% (for f'_c = 5458 psi)	-3.2% (for f'_c = 5458 psi)	Yes	3.7% (for f'_c = 3000 psi)

Table C-1. Evaluations With Sufficient Anticipated Margin
Electrical Tunnel 'B', Room CBST1

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
Slab	Flexure	Embedment & Splice Length Increased 17%	34%	No	34%
Slab	Out-of-Plane Shear	Concrete Shear Capacity Reduced 25%	44%	No	44%

C.6 Methods Employed to Gain Additional Margin

For the East wall (all elevations) ACI Code minimum horizontal reinforcement evaluation, additional margin was gained by accounting for the additional reinforcement installed beyond that credited in the calculation.

For the East wall (El. -20' to 0') in-plane shear evaluation, margin was found by accounting for an anticipated increase in the concrete shear capacity (see Section 4.0). This reduced the shear demand on the steel reinforcement. The anticipated increase in shear strength compared to that credited in the analysis is 25%, even after accounting for a potential reduction in strength from ASR. However, only a 2% increase in shear strength is required to meet the screening criteria from Section 4.0. For the West wall vertical flexure evaluation, additional margin was credited by using an accurate value for the depth of the rectangular compression block. A conservative value was used to calculate the margin documented in the calculation.

For the out-of-plane shear evaluations of the West wall, a simple 3-D finite element model of a plate fixed on all four sides was used to calculate a more accurate value of shear load. The calculated shear load was compared to the unreinforced shear limit for concrete with 3,000 psi compressive strength (calculated from $2\sqrt{f'_c}$). Based on the information in Reference 5, the anticipated increase in concrete shear strength beyond the ACI Code allowable will exceed the potential reduction in concrete shear capacity due to ASR.

For the in-plane shear evaluation of the East wall for elevations 0' to 21.5', no additional margin was able to be credited, but the demand on the lap splices was able to be reduced. Of the two rebar mats installed, only one mat was anchored using lap splices. The other rebar mat was anchored using rebar hooks, which are not expected to have reduced strength from ASR (see Section 4.0). Since there is margin documented in the calculation, the rebar anchored with hooks can carry additional load to reduce the demand on the rebar anchored with splices.

Table C-2. Review Results
'B' Electrical Tunnel, Room CBST1

Location: 'B' Electrical Tunnel - CBST1
Areas Affected by ASR: North, East, and West Walls and Floor Slabs - Below Grade (El 20' and below)
Calculations: C-S-1-10159, Rev. 0 and CD-20, Rev. 4
Drawings: 111340, 111342, 111343, 101345, FP11545

	As Documented															Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
Calc	Wall	Evaluation	Pages	Method	Loads	f'c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
CD-20	East	Horizontal Min Rebar	45	Hand Calc, ACI Code	N/A	N/A	N/A	N/A	A_s = 1.58 in²/ft, Note A	A_s = 1.44 in²/ft, Note A	8.9%	Yes	A_s = 1.44 in²/ft, Note A	8.9%	N/A	Embedment/ Splice Length	Yes	A_s = 3.12 in²/ft, Note A	6	54%	No
CD-20	East	Vertical Reinf., Flexure	46-47	Computer Run, Interaction Diagram	Note B	Note D	J	J	P_c = 150k M_c =120 ft-k Note T	P_u = 113.5k M_u =11.31 ft-k	24%, Note C	Yes	A_s = at most 1.27 in²/ft, E. F.	19%		Embedment/ Splice Length	No	A_s = 1.56 in²/ft, E. F.			No
CD-20	East	In Plane Shear - EL -20' to 0'	48-53	Computer Run, Hand Calc	Dead, Live, Seismic	3000	J	J	Rebar Spacing = 12"	Rebar Spacing = 14.42", Note F	16.8%	Yes	A_s = 3.46in², Notes A & U	16.8%	N/A	Embedment/ Splice Length	Yes	A_s = 4.16 in²/ft, Note A	7	At least 17%, Note L	No
CD-20	East	In Plane Shear EL 0' to 21.5'	48-53	Computer Run, Hand Calc	Dead, Live, Seismic	3000	J	J	A_s = 3.12 in²/ft, Note A	A_s = 2.75 in²/ft, Notes A & F	11.9%	Yes	A_s = 2.75 in²/ft, Note A	11.9%	N/A	Embedment/ Splice Length	Yes	A_s = 3.12 in²/ft, Note A	8	11.9%	No, Note E
CD-20	West (Sht. 4, Wall B)	Horizontal Reinf., Flexure	66	Table, Two-way slab	Hydro + Seismic	3000	N/A	N/A	A_s = 1.56 in²/ft, E. F.	A_s = 1.16 in²/ft E. F., Note G	26%	Yes	A_s = 1.16 in²/ft E. F.	26%	Edge (Mid-span)	Embedment/ Splice Length	No	A_s = 1.56 in²/ft, E. F.			No
CD-20	West (Sht. 4, Wall B)	Vertical Reinf., Flexure	67	Table, Two-way slab	Hydro + Seismic	3000	N/A	N/A	A_s = 1.00 in²/ft, E. F.	A_s = 0.85 in²/ft E. F., Note G	15%	Yes	A_s = 0.85 in²/ft, E. F.	15%	EL -20', mid-span	Embedment/ Splice Length	Yes	A_s = 1.00 in²/ft, E. F.	3	19%	No
CD-20	North (Sht. 4, Wall F)	Vertical Reinf., Flexure	75	Hand, One-way slab	Hydro + Seismic	N/A	N/A	N/A	A_s = 1.00 in²/ft E. F.	A_s = 0.40 in²/ft E. F., Note I	60%	Yes	A_s = 0.40 in²/ft, E. F.	60%	EL -20'	Embedment/ Splice Length	No	A_s = 1.00 in²/ft, E. F.			No
C-S-1-10159	West (Sht. 4 of Calc CD-20, Wall B)	Out-of-plane shear	3-4	Table, Two-way slab	Hydro + Seismic	4790	N/A	N/A	V_c = 26.85 k/ft	V_u = 25.19 k/ft, Note N	6.2%	No			N/A	Concrete Out-of-Plane Shear Capacity, Note W	Yes		1	3.7% with f'_c = 3000 psi	No

	As Documented																	Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
CD-20	West (Sht. 4, Wall B)	Out-of-plane shear	A4-A6	Computer Run, Hand Calc	Hydro + Seismic	5457.8	J	J	V_c = 29.46 k/ft	V_u = 30.41 k/ft,	-3.2%	No			N/A	Concrete Out-of-Plane Shear Capacity, Note W	Yes		1	3.7% with f'_c = 3000 psi	No
CD-20	Slab	Flexure	87	Hand, One-way slab, simply supported	Hydro + Dead + Seismic	N/A	N/A	N/A	A_s = 0.79 in^2/ft, E. F.	$A_{s(min)}$ = 0.52 in^2/ft, E. F., Note S	34%	Yes	$A_{s(min)}$ = 0.52 in^2/ft, E. F., Note S	34%	N/A	Embedment/ Splice Length	No	A_s = 0.79 in^2/ft, E F.			No
CD-20	Slab	Out of Plane Shear	C1	Hand Calc, One-way slab	Hydro + Dead + Seismic	3000	N/A	N/A	V_c = 48 kip/ft	V_u = 26.8 kip/ft	44%	No			N/A	Concrete Out of Plane Shear Capacity	No				No

Options to Increase Margin:

1. Create FEA model to better approximate load distribution, and recalculate shear load a distance 'd' away from the face of the support walls.
2. Not used.
3. Use a calculated value of 'a' (depth of equivalent rectangular stress block) rather than the conservative value assumed in CD-20.
4. Not used.
5. Not used.
6. Account for additional reinforcement above that credited in the analysis.
7. Account for increased concrete shear capacity based on anticipated concrete strength.
8. Since rebar on one face is anchored using lap splices and the other face uses standard hooks (of which the strength is not impacted by ASR), account for a reduced demand on the spliced rebar.

Specific Notes:

A. The rebar area presented is the total rebar area from both faces.
B. There are no details given about what loads were actually evaluated in the computer run, though Sheet 7 describes the limiting load cases.
C. Calculated based on the axial load capacity for a moment equal to M_u.
D. The material properties used to develop the interaction diagram were not provided.
E. The rebar on the inside face of the wall from elevation 0' to 1' and 2.75' to 17.5' is not anchored using splices (DWG FP11545). The rebar between elevations 1' and 2.75' are spliced, but there is three-way reinforcement surrounding the splice. Thus, splices vulnerable to reduced strength from ASR are only on the outside face of the wall. Since there is 11.9% margin for the inside face steel, it can carry additional load and reduce the demand on the outside face steel. The margin required to offset the lap splice strength reduction from ASR will be 17% - 11.9% = 5.1%.
F. Total shear load above EL 21.5' was calculated in CD-14.
G. Instead of triangular hydrostatic load, load was approximated as rectangular.
H. Plate was considered to be fixed on all sides.

Calculation No.	Prepared By	Checked By	Page: C-6
0326-0058-63	Kevin Getz	Ryan Maxwell	Revision: 1

I. Full height of wall is used (42') for determining if it is a one- or two-way slab, but the wall is intersected by a slab at the wall mid-height. The portion of the wall below the slab is the only portion of the wall with hydrostatic pressure and is 20' x 11.33'. In the calculation, the hydrostatic load was given a triangular load distribution along the horizontal span of the wall and the horizontal span was evaluated as a one-way slab. The hydrostatic load should be constant over the horizontal span; however, evaluating the wall as a one way slab with the triangular load distribution produces conservative results compared to a two-way slab analysis, which is more appropriate.
J. A computer analysis was used to calculate the loads on the structure. The concrete material properties used in the model were not specified in CD-20.
K. Not used.
L. If the concrete shear capacity is increased by 2%, the screening criterion for lap splices is met. The anticipated increase in shear strength is 25%, even after accounting for potential shear strength reduction from ASR, so the evaluation is expected to have sufficient margin to offset any adverse ASR effects.
M. Not used.
N. Wall was treated as fixed on three sides, pinned on top. This was so that a table for a triangular load distribution could be used.
O. Not used.
P. Not used.
Q. Not used.
R. Not used.
S. The minimum required reinforcement evaluation controls the rebar required for flexure, so the minimum reinforcement requirement is presented here.
T. The moment and axial force limits were each calculated assuming that the other load (force or moment) was equal to the demand value.
U. This number is not presented in calculation CD-20. Instead, it is calculated using the same methodology documented therein.
V. Not used.
W. Since this evaluation already accounted for an increase in the concrete compressive strength beyond 3000 psi (thus, increasing the concrete shear strength), there is an anticipated 25% reduction in the calculated concrete shear capacity (see Section 4.0).

D
RCA Tunnel, Unit 1 Tunnels

D.1 ASR Affected Areas

The West and East walls in the RCA Tunnel of Unit 1 show evidence of tight pattern cracking and active seepage. The areas evaluated for potential degradation by ASR are the north end of the East wall from elevation 0' to the ceiling and the West wall at the location of core bores RCAW 1-4.

D.2 Reviewed Calculations

Calculation SG-1 Revision 2, CD-10 Revision 1, and WB-69 Revision 7 were reviewed to determine the documented margin for the West and East walls of the RCA Tunnel. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table D-3.

D.3 Calculation General Methodology

Calculation SG-1, Revision 2 evaluated the walls of the RCA Tunnel in the Non-Essential Switchgear Room. The walls were conservatively designed based on the larger loads determined for the RCA Tunnel which runs from the Emergency Feed Water Building of Unit #2 to the Administration Building. These loads were referenced from calculation EM-7. The loads on the tunnel walls were calculated by hand and then used in a computer analysis that modeled the tunnel as a single bay portal bent, fixed at the base. Once the loads were calculated, the walls were evaluated by hand to ACI Code limits (Reference 3).

Calculation CD-10, Revision 1 evaluated the walls and mat slabs in the RCA Tunnel under the Control Building. An interaction diagram was used to evaluate the walls. The moments on the walls were determined by modeling the walls as one-way slabs and the axial loads were calculated from the loads from the above slab.

Calculation WB-69, Revision 7 evaluated the walls and slabs for the pipe tunnel of the Waste Processing Tank Farm. The tunnel was modeled as a two-dimensional rigid frame and was analyzed by a computer. The reinforcement in the walls was evaluated against the minimum required by code.

D.4 Evaluations without Sufficient Anticipated Margin

The evaluations in Table D-1 did not meet the screening criteria described in Section 4.0. Unlike the other areas evaluated in the calculation, the reinforcement in this portion of the RCA tunnel consists of No. 6 bars. As discussed in Section 4.0, bars smaller than No. 7 require 40% margin to meet screening criteria for development and splice length. No simplified methods for extracting additional margin were able to be employed to meet these criteria. For the two flexure evaluations, more margin may be found from re-evaluating the structure with a detailed computer model, but this effort was beyond the scope of this review and the potential benefit from such efforts is unknown. For the three evaluations in Table D-1, the "Anticipated Margin" column reports the documented margin.

Table D-1. Evaluations Without Sufficient Anticipated Margin
RCA Tunnel, Unit 1 Tunnels

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
NE Corner of Tunnel	Vertical Reinf: Flexure and Compression	Embedment & Splice Length Increased 40%	18%	No	18%
NE Corner of Tunnel	Horizontal Reinf: Shear	Embedment & Splice Length Increased 40%	39%	No	39%
West Wall (Control Bldg) - Core Bore RCAW-1&2	Flexure and Tension	Embedment & Splice Length Increased 40%	18%	No	18%

D.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table D-2 satisfied the screening criteria described in Section 4.0. The following two evaluations are in a different portion of the RCA tunnel than the evaluations discussed in Section D.4. The reinforcement in this portion of the RCA tunnel consists of No. 8 bars. For both cases, the "Anticipated Margin" column reports the documented margin.

Table D-2. Evaluations With Sufficient Anticipated Margin RCA Tunnel, Unit 1 Tunnels

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
West Wall (Control Bldg) - Core Bore RCAW-3&4	Vertical Minimum Reinf	Embedment & Splice Length Increased 17%	72%	No	72%
West Wall (Control Bldg) - Core Bore RCAW-3&4	Horizontal Minimum Reinf.	Embedment & Splice Length Increased 17%	54%	No	54%

D.6 Methods Employed to Gain Additional Margin

For the evaluations in Table D-1, no additional margin could be gained to meet the screening criteria outlined in Section 4.0. It should be noted that the loads used on the walls were calculated based on a different portion of the RCA tunnel, which runs from the Emergency Feed Water Building of Unit #2 to the Administration Building. Reevaluating the RCA Tunnel with the actual loads applicable to this portion of the RCA tunnel could identify additional margin.

For the evaluations in Table D-2, no additional margin needed to be gained to meet the screening criteria outlined in Section 4.0.

Table D-3. Review Results
RCA Tunnel

Location: RCA Tunnel
Areas Affected by ASR: NE Wall @ El. 0' & All Cored Walls
Calculations: SG-1 Rev. 2, CD-10 Rev. 1, WB-69 Rev. 7
Drawings: 112000, 111340, 111342, 111814, 111821

As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
SG-1	NE Corner of Tunnel	Vertical Reinf: Flexure and Compression	58-60	Hand Calc	(I)	3000	H	H	P_c = 26.5 k	P_u = 21.71 k	18% (G)	Yes	A_s = 0.36 in^2/ft E.F. (D)	18%	--	Development / Splice Length	Yes, (L)	A_s = 0.44 in^2/ft E. F.	--	18%	Yes, (L)
SG-1	NE Corner of Tunnel	Horizontal Reinf: Shear	62-63	FEM, Hand Calc	(I)	3000	H	H	v_c = 108 psi A_s = 0.88 in^2/ft	v_u = 138 psi $A_{s(min)}$ = 0.54 in^2/ft, (J)	39%	Yes	A_s = 0.54 in^2/ft E. F.	39%	--	Development / Splice Length	Yes, (L)	A_s = 0.44 in^2/ft E. F.	--	39%	Yes, (L)
CD-10	West Wall (Control Bldg) - Core Bore RCAW-1&2	Flexure and Tension	7	Interaction Diagram	Hydro, DL, LL, Seismic	3000	N/A	N/A	Axial: 22 k (ten.) Moment: 50 ft-k (E) A_s = 0.44 in^2/ft E. F.	Axial: 15.21 k (comp.) Moment. 18.6 ft-k $A_{s(min)}$ = 0.36 in^2/ft E.F.	18% (B, K)	Yes	A_s = 0.36 in^2/ft E.F. (F)	18%	--	Development / Splice Length	Yes, (L)	A_s = 0.44 in^2/ ft E.F.	--	18%	Yes, (L)
WB-69	West Wall (Control Bldg) - Core Bore RCAW-3&4	Vertical Minimum Reinf	41, 45, M13	Hand Calc	N/A	N/A	N/A	N/A	A_s = 0.79 in^2 /ft E. F.	$A_{s(min)}$ = 0.22 in^2/ft E. F. (C)	72%	Yes	$A_{s(min)}$ = 0.22 in^2/ft E. F. (C)	72%	--	Development / Splice Length	No, (A)	A_s = 0.79 in^2 /ft E. F.	--	--	No
WB-69	West Wall (Control Bldg) - Core Bore RCAW-3&4	Horizontal Minimum Reinf.	41, 45, M13	Hand Calc	N/A	N/A	N/A	N/A	A_s = 0.79 in^2 /ft E. F.	$A_{s(min)}$ = 0.36 in^2/ft E. F. (C)	54%	Yes	$A_{s(min)}$ = 0.36 in^2/ft E. F. (C)	54%	--	Development / Splice Length	No, (A)	A_s = 0.79 in^2 /ft E. F.	--	--	No

Specific Notes:

A. Margin for reinforcement required is greater than screening criteria of 17% for development and splice length.
B. Conservatively assumed all of the moment from the mat spanning the tunnel is transferred to the rest of the mat and the shear is transferred to the wall - Sheet 6 of 10.

C. The demand loads for this section of the wall are not clearly defined. The calculation states that nominal reinforcement is adequate based on inspection. From page M13 of this calculation, nominal reinforcement is defined as 0.0025×b×t for horizontal reinforcement and 0.0015×b×t for vertical reinforcement. This is the origin of the required reinforcement for this section.

D. Value is based on a scoping calculation to determine the minimum area of reinforcement required to resist the applied loads. This value is greater than the minimum required by the Code.

E. The moment and axial force limits were each calculated assuming that the other load (force or moment) was equal to the demand value.

F. Point is within the interaction diagram curve for the #6 @ 12". Based on a scoping calculation, the area of steel required to resist the moment and axial force will be less than the minimum requirement.

G. Margin based on axial load.

H. These properties were not used in the evaluation of the wall as documented in the indicated calculation. The calculation that developed the loads used in this calculation, EM-7, gives an E of 3.000E+06 psi, but does not state the value of ν.

I. The loads used in the calculation were referenced from calculation EM-7. The loads considered in EM-7 are: dead, hydrostatic, soil pressure, compaction and surcharge, and seismic (OBE).

J. Based on minimum required horizontal reinforcement for walls, 0.0025×b×t.

K. Margin based on $A_{s(min)}$.

L. Margin for reinforcement required is less than the screening criteria of 40% for development and splice length for bars smaller than No. 7.

E

Diesel Generator Building, Room DG102

E.1 ASR Affected Areas

The East wall of the Diesel Generator Building, room DG102, shows evidence of cracking, potentially by ASR. The area evaluated for potential degradation by ASR is the East wall in DG102 from elevation (-)16' up to grade. Though the West wall was reviewed to document existing margin, no indications of ASR were reported in the walkdown report for this wall in room DG102 (Reference 7).

E.2 Reviewed Calculations

Calculation CD-18, Revision 5 was reviewed to determine the documented margin for the East wall of DG102. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table E-3.

E.3 Calculation General Methodology

Calculation CD-18, Revision 5 evaluated the walls below grade and the mat foundation for the Diesel Generator Building. The walls and slabs were designed as vertical frame sections, in the North-South and East-West directions. The moment distribution method was used to calculate the final moments acting on the walls slabs. The in-plane moments were calculated by hand and the vertical tension and compression loads on the walls were determined with a computer analysis. The required reinforcement was determined based on moment diagrams and interaction diagrams for the walls.

E.4 Evaluations without Sufficient Anticipated Margin

A single evaluation in Table E-1 did not meet the screening criteria for lap spice/embedment length described in Section 4.0. Additional margin was calculated above what was documented in the calculation, but the anticipated margin still did not meet the screening criteria of Section 4.0. Section E.6 describes the method used to extract the additional margin documented in the table.

Table E-1. Evaluations Without Sufficient Anticipated Margin
Diesel Generator Building, Room DG102

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
East	Flexure	Embedment & Splice Length Increased 17%	5.3%	Yes	10%

E.5 Evaluations with Sufficient Anticipated Margin

Table E-2 contains the evaluations for the West and East walls that met the screening criteria of Section 4.0. The evaluations of the West wall are not considered an operability concern for ASR, regardless of margin.

The evaluations of the East wall satisfied the screening criteria described in Section 4.0. In one instance, the margin documented in the calculation did not meet the screening criteria, so additional margin was extracted from the evaluation by one of the methods described in Section 5.2. The additional margin was sufficient to satisfy the screening criteria. The applicable evaluations for which additional margin was calculated are indicated in the following table and Section E.6 describes the methods used to extract the additional margin. For the cases in which the documented margin met the screening criteria, the "Anticipated Margin" column reports the documented margin.

Table E-2. Evaluations With Sufficient Anticipated Margin
Diesel Generator Building, Room DG102

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
West	Flexure	N/A (ASR not observed in West Wall)	16.3%	No	16.3%
West	Flexure	N/A (ASR not observed in West Wall)	7.7%	Yes	13%

Table E-2. Evaluations With Sufficient Anticipated Margin
Diesel Generator Building, Room DG102

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
West	Flexure	N/A (ASR not observed in West Wall)	25%	No	25%
West	Flexure	N/A (ASR not observed in West Wall)	23%	No	23%
West	Flexure	N/A (ASR not observed in West Wall)	20%	No	20%
West	In Plane Shear	N/A (ASR not observed in West Wall)	8.9%	Yes	8.9% - 30% (See Table E-3)
West	Flexure	N/A (ASR not observed in West Wall)	17.7%	No	17.7%
East	In Plane Shear	Embedment & Splice Length Increased 17%	8.9%	Yes	30%
East	Flexure	Embedment & Splice Length Increased 17%	20%	No	20%
East	Flexure	Embedment & Splice Length Increased 17%	25%	No	25%
East	Out of Plane Shear	Concrete Shear Capacity Reduced 25%	-1.2%	Yes	30%

E.6 Methods Employed to Gain Additional Margin

The evaluation of the East wall for flexure, listed in Table E-1, did not meet the screening criteria described in Section 4.0. Additional margin was identified by using an interaction diagram which credits the compression steel.

The evaluation of the East wall for in-plane shear, listed in Table E-2, identified considerable margin by crediting the staggered horizontal rebar splices, which reduces the required lap splice length. The evaluation of the East wall for out-of-plane shear identified margin based on a scoping calculation with a 3D solid finite element model of a wall with the same dimensions and loads as the East wall. The wall was fixed on all sides and was evaluated for hydrostatic, seismic, static, and dynamic soil loads.

Although the evaluations of the West wall for flexure and in-plane shear, listed in Table E-2, were considered acceptable, additional margin was identified in some cases. The evaluation of the West wall in flexure identified additional margin by taking into account the compressive load acting on the West wall due to the self-weight of the wall and the weight of a portion of the slab above the West wall. This compressive load, along with the moment on the wall, was used in an interaction diagram to determine the area of reinforcement required. The in-plane shear evaluation of the West wall identified margin in portions of the wall because the horizontal rebar in the wall above elevation (-)8.5' was staggered and the horizontal rebar in the wall below (-)12' on the inside face of the wall was anchored with hooks at both ends.

Table E-3. Review Results
Control and Diesel Generator Building, Room DG102

Location: Control and Diesel Generator Bldg - DG102
Areas Affected by ASR: East Wall - EL -16'
Calculations: CD-18, Rev. 5
Drawings: 101381, 101382, 111384, 111385, FP10288, FP13396

As Documented																Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
CD-18	West	Flexure	28	FEM, Moment distribution method	Hydro , Static Soil Pr., surcharge, compaction, and seismic (OBE)	1	N/A	N/A	A_s = 2.08 in^2/ft E.F.	$A_{s(reqd)}$ = 1.61 in^2/ft E.F. $A_{s(min)}$ = 1.74 in^2/ft E.F., (W)	16.3% (based on min)	Yes	$A_{s(reqd)}$ = 1.61 in^2/ft E.F. $A_{s(min)}$ = 1.74 in^2/ft E.F., (W)	16.3%	El. 20' (A)	Embedment/ Splice Length	No, (D)	A_s = 2.08 in^2/ft E.F.			No, (D, III)
CD-18	West	Flexure	56	FEM, Moment distribution method	Hydro., Static Soil Pr., surcharge, compaction (No Seismic)	1	N/A	N/A	A_s = 2.08 in^2/ft E.F.	$A_{s(reqd)}$ = 1.92 in^2/ft E.F. $A_{s(min)}$ = 1.74 in^2/ft E.F., (W)	7.7% (based on required)	Yes	$A_{s(reqd)}$ = 1.92 in^2/ft E.F. $A_{s(min)}$ = 1.74 in^2/ft E.F., (W)	7.7%	El. 21'-6" (A)	Embedment/ Splice Length	Yes	A_s = 2.08 in^2/ft E.F.	1	13%	No, (H, III)
CD-18	West	Flexure	62	Interaction Diagram, Computer Model	Hydro., Static Soil Pr., surcharge, compaction, and seismic (OBE)	1	N/A	N/A	Tension: Pu = 130 k Mu = 275 ft-k (O)	Tension: Pu = 61.1 k Mu = 197 ft-k (N)	28% (moment)	Yes	A_s = 1.56 in^2/ft E.F., (L)	25%	El. 20' (A)	Embedment/ Splice Length	No, (D)	A_s = 2.08 in^2/ft E.F.	--	--	No, (D, III)
CD-18	West	Flexure	63-64	Interaction Diagram, FEA	Hydro., Static Soil Pr., surcharge, compaction, and seismic (OBE), seismic soil pressure due to surcharge	1	N/A	N/A	Tension: Pu = 120 k Mu = 275 ft-k (O)	Tension: Pu = 61.1 k Mu = 204.3 ft-k (N)	26% (moment)	Yes	A_s = 1.60 in^2/ft E.F., (L)	23%	--	Embedment/ Splice Length	No, (D)	A_s = 2.08 in^2/ft E.F.	--	--	No, (D, III)

As Documented																Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
CD-18	West	Flexure	76A-D, 62	Interaction Diagram, Computer Model	Hydro., Static Soil Pr., surcharge, compaction, and seismic (OBE), seismic soil pressure due to surcharge, dynamic soil pressure	I	N/A	N/A	Tension: Pu = 110 k Mu = 275 ft-k (O)	Tension: Pu = 61.1 k Mu = 217 ft-k (N)	21% (moment)	Yes	A_s = 1.67 in^2/ft E.F. (L, X)	20%	--	Embedment/ Splice Length	No	A_s = 2.08 in^2/ft E.F.			No, (III)
CD-18	West	In Plane Shear	76	Computer Model & ACI Code	Dead Load, Live Load, and seismic (OBE)	I	N/A	N/A	A_s = 0.79 in^2/ft	$A_{s(reqd)}$ = 0.38 in^2/ft $A_{s(min)}$ = 0.72 in^2/ft E.F.	8.9%	Yes	$A_{s(reqd)}$ = 0.38 in^2/ft $A_{s(min)}$ = 0.72 in^2/ft E.F.	8.9%	--	Embedment/ Splice Length	Yes	A_s = 0.79 in^2/ft E.F.	2	8.9% (U) 30% (V)	No, (G, III)
CD-18	West	Flexure	76A-D	FEM, Moment distribution method	Hydro., Static Soil Pr., surcharge, compaction, and seismic (OBE), seismic soil pressure due to surcharge, dynamic soil pressure	I	N/A	N/A	M_c = 376.8 ft-k	M_u = 310 ft-k	17.7%	Yes	--	--	--	Embedment/ Splice Length	No, (D)	--	--	--	No, (E, D, III)
CD-18	East	In Plane Shear	76	Computer Model & ACI Code	Dead Load, Live Load, and seismic (OBE)	I	N/A	N/A	A_s = 0.79 in^2/ft E.F. (C)	$A_{s(min)}$ = 0.72 in^2/ft E.F.	8.9%	Yes	$A_{s(min)}$ = 0.72 in^2/ft E.F.	8.9%	--	Embedment/ Splice Length	Yes	A_s = 0.79 in^2/ft E.F.	2	30%	No
CD-18	East	Flexure	35	FEM, Moment distribution method	Hydro., Static Soil Pr., surcharge, compaction, and seismic (OBE)	I	N/A	N/A	A_s = 2.08 in^2/ft E.F.	$A_{s(reqd)}$ = 1.67 in^2/ft E.F. $A_{s(min)}$ = 1.50 in^2/ft E.F.	20%	Yes	$A_{s(reqd)}$ = 1.67 in^2/ft E.F. $A_{s(min)}$ = 1.50 in^2/ft E.F.	20%	El.(-)17'-9" (B)	Embedment/ Splice Length	No, (F)	A_s = 2.08 in^2/ft E. F.	--	--	No
CD-18	East	Flexure	56	FEM, Moment distribution method	Hydro., Static Soil Pr., surcharge, compaction (No Seismic)	I	N/A	N/A	A_s = 2.08 in^2/ft E.F.	$A_{s(reqd)}$ = 1.97 in^2/ft E.F. $A_{s(min)}$ = 1.50 in^2/ft E.F.	5.3%	Yes	$A_{s(reqd)}$ = 1.97 in^2/ft E.F. $A_{s(min)}$ = 1.50 in^2/ft E.F.	5.3%	El. 21'-6"	Embedment/ Splice Length	Yes	A_s = 2.08 in^2/ft E.F.	1	10%	Yes

Calculation No.	Prepared By	Checked By	Page: E-7
0326-0058-63	Ryan Minil	Kevin Gotz	Revision: 1

As Documented																Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
CD-18	East	Flexure	66	Interaction Diagram, Computer Model	Hydro., Static Soil Pr., surcharge, compaction, and seismic (OBE)	I	N/A	N/A	Tension: P_c = 100 k M_c = 350 ft-k (O)	Tension: P_u = 0 k M_u = 180 ft-k (N)	49% (moment)	Yes	A_s = 1.56 in^2/ft E.F. (S)	25%	El. (-) 17'-9" (B)	Embedment/ Splice Length	No	A_s = 2.08 in^2/ft E.F.	--	--	No
CD-18	East	Out-of-Plane Shear	A9 (J)	FEA Model	Hydro., Static Soil Pr., surcharge, and compaction	I	(Z)	(Z)	V_c = 41.3 kip/ft	V_c = 41.8 kip/ft	-1.2%	No	N/A	N/A	N/A	Concrete Out of Plane Shear Capacity	Yes		3	30% (Y)	No

Options to Increase Margin:

1. Use an interaction diagram to evaluate, considering the compressive force available when seismic load is absent in the lateral direction (will include potential reduction in deadweight load due to vertical seismic acceleration).
2. Reduction for staggered rebar splices.
3. Evaluate the East wall explicitly with a 3D finite element model.

Specific Notes:

A. Max moment - refer to Sheet 27
B. Max moment - refer to Sheet 34
C. Only the West wall was evaluated. Calculation states that the East wall does not extend above grade and therefore should use the same steel as the West wall.
D. Evaluation is bounded by the flexure evaluation of the West Wall including the revised dynamic soil pressure.
E. Dynamic soil pressure is increased from 208.8 to 375 lb/ft. Static soil pressure is decreased from 720 to 625 lb/ft.
F. Evaluation is bounded by the flexure evaluation of the East Wall using the interaction diagram.
G. The only portion of the wall that has less margin than the anticipated reduction in lap splice strength due to ASR is from elevation (-)12' to (-)8.5'.
H. Calculation CD-18 assumes that a seismic event and the placement of the diesel generator between column lines 7 & 9 and A & E will not occur at the same time.
I. Sheet 3 - fc' = 3000 psi.
J. The evaluation for out-of-plane shear for the East wall was not explicitly included in calculation CD-18. The calculation analyzed the North wall only, as it was bounding. The results for the North wall are documented here.
K. Not used.
L. Estimation based on interaction curve given on Sheet 62.
M. Not used.
N. A technical issue with the load demand was identified during review and has been communicated to the plant. The technical issue is addressed outside of this review. The margin presented is based on the methodology documented in CD-18.
O. The moment and axial force limits were each calculated assuming that the other load (force or moment) was equal to the demand value.

P. Not used.
Q. Not used.
R. Not used.
S. This is conservative, the point lies inside of the #11@12 line on the interaction diagram.
T. Not used.
U. The horizontal rebar is not staggered from elevation (-)16' to (-)8.5'. However, the rebar on the inside face of the wall from elevation (-)16' to (-)12' are not anchored using splices. No additional margin could be gained for the wall in this region because rebar splices are not staggered in this section, but the margin required to offset the lap splice strength reduction from ASR will be 17% - 8.9% = 8.1% where rebar is spliced on one face only.
V. The horizontal rebar is staggered from elevation (-)8.5' to 17.5'. Since the splices are staggered, they are classified as Class B splices. The required splice length for Class B splices is $1.3l_d$ as opposed to the $1.7l_d$ splice that exists in the design. Additional margin is gained by accounting for the excess splice length.
W. The ratio for minimum reinforcement for flexure is used (0.0033). The ratio for walls should be used (0.0015), which would result in a minimum area of reinforcement of $A_{s(min)} = 0.86$ in^2/ft (total from both faces).
X. The reinforcement limit was confirmed by an independent scoping calculation.
Y. The calculated demand for the East wall presented herein was determined based on a scoping calculation with a 3D finite element model of a wall fixed on four sides under hydrostatic pressure, static soil pressure, dynamic soil pressure, and seismic acceleration.
Z. The wall was evaluated with a finite element model. The elastic modulus and Poisson's Ratio used in the evaluation were not provided in CD-18.

General Notes:

I. East Wall flexure with dynamic soil pressure not explicitly evaluated in calculation. Wall was qualified by comparison to the re-evaluation of the West Wall.
II. Only the North wall was evaluated for out of plane shear. North wall was qualified without crediting steel. Wall was locally overstressed by 0.5 kip/ft, but was judged to be OK. (Sheet A9 of A9)
III. ASR degradation on the West wall was not documented during the walkdown evaluation of DG102. The anticipated margin for the West wall should not be compared against the screening criteria of Section 4.0.

F

Primary Auxiliary Building, Room PB205

F.1 ASR Affected Areas

The South and East walls of the Primary Auxiliary Building, room PB205, have observed pattern cracking and horizontal cracks. The areas evaluated for potential degradation by ASR are the South wall and the south portion of the East wall in PB205 from elevation (-)6' to 3'.

F.2 Reviewed Calculations

Calculation PB-20, Revision 4 and WB-82, Revision 1 (see the assumption in Section 3.1) were reviewed to determine the documented margin for the East and South walls of PB205. After a review of the drawings, it was determined that the East wall of PB205 is also the West wall of the Primary Auxiliary Building. Therefore, calculation PB-20, which evaluates the West wall of the Primary Auxiliary Building, was reviewed to document the margin for the East wall of PB205. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table F-2.

F.3 Calculation General Methodology

Calculation PB-20, Revision 4 evaluated the West wall of the Primary Auxiliary Building based on a finite element analysis. The required area of reinforcement was calculated using the loads from the finite element analysis. Calculation PB-20 states that the West wall of the Primary Auxiliary Building is subject to additional loads, which were evaluated in calculation PB-65. After review of this calculation, it was determined that the loads were from pipe supports located in areas that did not affect the evaluation of the walls affected by ASR.

Calculation WB-82, Revision 1 evaluated the walls of the Waste Processing Building below elevation 25'. The walls were designed using loads from a computer model. The most limiting load combination was used to size the vertical and horizontal reinforcement.

F.4 Evaluations without Sufficient Anticipated Margin

None.

F.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table F-1 satisfied the screening criteria described in Section 4.0. For all the cases, the "Anticipated Margin" column reports the documented margin.

Table F-1. Evaluations With Sufficient Anticipated Margin
Primary Auxiliary Building, Room PB205

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
South Wall	Vertical: Flexure and Tension	Embedment & Splice Length Increased 17%	34%	No	34%
South Wall	Horizontal: Flexure and Tension & In-Plane Shear	Embedment & Splice Length Increased 17%	51%	No	51%
West Wall	Vertical: Flexure and Tension (Element #124)	Embedment & Splice Length Increased 17%	38%	No	38%
West Wall	Vertical: Flexure and Tension (Element #125)	Embedment & Splice Length Increased 17%	23.7%	No	23.7%
West Wall	Vertical: Flexure and Compression (bounding)	Embedment & Splice Length Increased 17%	71%	No	71%
West Wall	Horizontal: Flexure and Tension & In-Plane Shear	Embedment & Splice Length Increased 17%	34%	No	34%

F.6 Methods Employed to Gain Additional Margin

The documented margin for the evaluations of the South and East walls of PB205 met the screening criteria of Section 4.0. No additional margin needed to be calculated.

Table F-2. Review Results
Primary Auxiliary Building, Room PB205

Location: Primary Auxiliary Building - PB205
Areas Affected by ASR: South Wall, East Wall (South Portion), El. (-) 6'
Calculations: PB-20, Rev. 4 & WB-82, Rev. 1
Drawings: 101512, 101514, 101520, 111814, 111816, 111819, 111821, 805061, 805069
Note: *The East Wall of interest is the West Wall of the Primary Auxiliary Building in Calc PB-20*

As Documented																Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
WB-82	South Wall	Vertical: Flexure and Tension	23	Finite Element Computer Output, Hand Calc	Load Eq. 3	3000	I	I	$A_s = 1.56$ in²/ft E.F.	$A_s = 0.95$ in²/ft E. F.	34%	Yes	$A_s = 0.95$ in²/ft E. F.	34%	--	Development/ Splice Length	No, (J)	$A_s = 1.56$ in²/ft E. F.	--	--	No
WB-82	South Wall	Horizontal: Flexure and Tension & In-Plane Shear	24-25	Finite Element Computer Output, Hand Calc	Load Eq. 3	3000	I	I	$A_s = 1.56$ in²/ft E. F.	$A_s = 0.76$ in²/ft E. F. (H)	51%	Yes	$A_s = 0.76$ in²/ft E. F. (H)	51%	--	Development/ Splice Length	No, (J)	$A_s = 1.56$ in²/ft E. F.	--	--	No
PB-20	West Wall	Vertical: Flexure and Tension (Element #124)	92	Finite Element Computer Output, Hand Calc	C	3000	I	I	$A_s = 3.12$ in²/ft E. F.	$A_s = 1.93$ in²/ft E. F.	38%	Yes	$A_s = 1.93$ in²/ft E. F.	38%	--	Development/ Splice Length	No, (J)	$A_s = 3.12$ in²/ft E. F.	--	--	No
PB-20	West Wall	Vertical: Flexure and Tension (Element #125)	93	Finite Element Computer Output, Hand Calc	C	3000	I	I	$A_s = 1.56$ in²/ft E. F.	$A_s = 1.19$ in²/ft E. F.	23.7% (A)	Yes	$A_s = 1.19$ in²/ft E. F.	23.7%	--	Development/ Splice Length	No, (J)	$A_s = 1.56$ in²/ft E. F.	--	--	No
PB-20	West Wall	Vertical: Flexure and Compression (bounding)	100-102	Finite Element Computer Output, Hand Calc	D	3000	I	I	$M_c = 147$ ft-k	$M_u = 43.2$ ft-k	71%	Yes	Note G	--	--	Development/ Splice Length	No, (G)	$A_s = 1.56$ in²/ft E. F	--	--	No
PB-20	West Wall	Horizontal. Flexure and Tension & In-Plane Shear	117-123	Finite Element Computer Output, Hand Calc	D	3000	I	I	$A_s = 1.56$ in²/ft E. F.	$A_s = 1.03$ in²/ft E. F. (H)	34%	Yes	$A_s = 1.03$ in²/ft E. F. (H)	34%	--	Development/ Splice Length	No, (J)	$A_s = 1.56$ in²/ft E. F.	--	--	No

	As Documented																Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
PB-20	West Wall - Corbel (K)	Tension & Shear Reinforcement	A4-5	Hand Calc	E	--	--	--	--	--	--	--	--	--	--	--	No, (K)	--	--	--	No

General Notes:

1. Calculation PB-20 referenced calculation PB-65 for additional loads on the West wall. Review of PB-65 has shown that the revised loads were due to pipe supports, and the evaluated elevations were outside the scope of the evaluation.

Specific Notes:

A. Around half of the element has double the amount of listed vertical reinforcement. However, elements 126 and 127, which are the elements nearest to where ASR was observed only have the reinforcement listed.
B. Not used.
C. Case 2: Maximum tension with corresponding moment (U2 = 1.2D + 1.9Feqo).
D. Case 1: Maximum compression with corresponding moment (U1 = 1.4D + 1.7L + 1.9Feqo).
E. Loads from Calc WB-62.
F. Not used.
G. Based on the margin presented for the load, the evaluation with maximum tension and the corresponding moment is bounding in terms of reinforcement design. The load case evaluated is also very conservative for the elements that are affected by ASR (126 and 127).
H. Requirement from the combination of both in-plane shear and horizontal flexure reinforcement calculations.
I. Computer model was used to extract loads on walls. It is not known what concrete elastic modulus and Poisson's ratio were used in the model.
J. Margin for reinforcement required is greater than screening criteria of 17% for development and splice length.
K. Corbels are located between El. 1' to 2'. No indications of ASR degradation on the actual corbels.

Calculation No.	Prepared By	Checked By	Page: G-1
0326-0058-63	Kevin Götz	Ryan Marcil	Revision: 1

G

Primary Auxiliary Building Mechanical Penetration, Room MF102

G.1 ASR Affected Areas

The thickened portion of the North wall (8-ft thick below elevation (-)26' and 14-ft thick above this elevation) shows evidence of cracking. Although the cracking is not necessarily due to ASR, the area between El. (-)34'-6" and (-)26' is evaluated for potential concrete degradation due to ASR.

G.2 Reviewed Calculations

Calculation EM-31, Revision 6 was reviewed. Only the relevant information pertaining to the evaluation of the ASR-affected area listed above was reviewed as part of this effort. The results of the review are presented in Table G-2.

G.3 Calculation General Methodology

The thickened portion of the north wall (the area potentially affected by ASR) is only evaluated for the minimum reinforcement requirement from the ACI Code. Loads on the wall from external loads are very small, so the minimum reinforcement requirement is limiting.

G.4 Evaluations without Sufficient Anticipated Margin

None.

G.5 Evaluations with Sufficient Anticipated Margin

The margins documented in the reviewed calculation for all evaluations satisfied the screening criteria described in Section 4.0. Note that the calculation did not document the amount of margin, but it was stated that the reinforcement in the wall would satisfy the minimum reinforcement requirement. For the results tables and the summary tables presented herein, the amount of minimum reinforcement required per the ACI Code is calculated and compared to the reinforcement present in the wall. The "Anticipated Margin" column and the "Documented Margin" columns report this value.

Table G-1. Evaluations With Sufficient Anticipated Margin
Primary Auxiliary Building Mechanical Penetration, Room MF102

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
North (8-ft thick portion)	Vertical Minimum Reinforcement	Embedment & Splice Length Increased 17%	67%	No	67%
North (8-ft thick portion)	Horizontal Minimum Reinforcement	Embedment & Splice Length Increased 17%	31%	No	31%

G.6 Methods Employed to Gain Additional Margin

No recovery of analysis conservatism was required.

Table G-2. Review Results
Primary Auxiliary Building Mechanical Penetration, Room MF102

Location: PAB Mechanical Penetration Area - MF102
Areas Affected by ASR: North Wall (Column near NE Corner of Room), El. -34'
Calculations: EM-31, Rev. 6
Drawings: 101625, 101626, 101627, 101628, 101632

Calc	As Documented															Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
	Wall	Evaluation	Pages	Method	Loads	f'c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
EM-31	North (Wall 207, 8ft thick portion)	Vertical Minimum Rebar	33	Note A	N/A	N/A	N/A	N/A	A_s = 3.12 in^2/ft, E. F., Note B	A_s = 1.04 in^2/ft, E. F., Note C	67%	Yes	A_s = 1.04 in^2/ft, E. F., Note C	67%	N/A	Embedment/ Splice Length	No	A_s = 3.12 in^2/ft, E. F., Note B			No
EM-31	North (Wall 207, 8 ft thick portion)	Horizontal Minimum Rebar	33	Note A	N/A	N/A	N/A	N/A	A_s = 2.08 in^2/ft, E. F., Note B	A_s = 1.44 in^2/ft, E. F., Note D	31%	Yes	A_s = 1.44 in^2/ft, E. F., Note D	31%	N/A	Embedment/ Splice Length	No	A_s = 2.08 in^2/ft, E. F., Note B			No

Options to Increase Margin:

1. Not used.

Specific Notes:

A. The calculation judges that minimum reinforcement requirements from the ACI code are sufficient for the thickened portion of the wall. The actual calculation of the quantity of rebar required to meet the minimum requirements is not performed. The calculation is performed as part of this review and the results are presented in the table.
B. From drawing 101627.
C. The minimum reinforcement is calculated based on $\rho = 0.0018$. This requirement is more conservative than the requirement for walls ($\rho = 0.0015$).
D. The minimum reinforcement is calculated based on $\rho = 0.0025$. This requirement is applicable to walls, and it is more conservative than the requirement for temperature and shrinkage reinforcement from Reference 3, Section 7.13 ($\rho = 0.0018$).

Calculation No.	Prepared By	Checked By	Page: H-1
0326-0058-63	Kevin Getz	Ryan Maisel	Revision: 1

H

Electrical Tunnel 'B', Room EF101

H.1 ASR Affected Areas

Pattern cracking and discoloration has been observed on the North and South Walls of EF101 between elevations (-)20' and (-)2'. These walls and the floor slab at elevation (-)20' are evaluated for potential degradation by ASR.

H.2 Reviewed Calculations

Calculations EF-4, Revision 9 and EF-11, Revision 1 were reviewed. Only the relevant information pertaining to the evaluation of the ASR-affected walls listed above was reviewed as part of this effort. The calculation for the North wall was already reviewed and the results were previously presented in Appendix B, so only the results of the South wall evaluation are presented herein. Note that the evaluation of the South wall in EF-4, Revision 9 has been superseded by calculation EF-11, Revision 1.

The calculation containing the design basis evaluation of the floor slab was not available for review. Thus, no operability judgments are made about the floor slab of EF101 herein. The results of the review are presented in Table H-2.

H.3 Calculation General Methodology

For the evaluation of the South Wall in calculation EF-11, the deformation of the West Wall from north-south direction design basis loads is calculated, and the South wall is assumed to displace along the same profile. The deformation of the West Wall is calculated from a combination of bending and shear behavior. The stresses in the South Wall are then calculated from the derived displacement. For the East-West direction loads, the South Wall is assumed to behave like a cantilever, though a reduced effective length is used to account for the reduction in moment that would occur with a fixed-fixed end condition.

H.4 Evaluations without Sufficient Anticipated Margin

None.

H.5 Evaluations with Sufficient Anticipated Margin

All evaluations of the South wall satisfied the screening criteria described in Section 4.0. For these cases, the documented margin met the screening criteria, so the "Anticipated Margin" column reports the documented margin.

Table H-1. Evaluations With Sufficient Anticipated Margin Electrical Tunnel 'B', Room EF101

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
South	Vertical Reinforcement, El. (-)20' to 0'	Embedment Length/Splice Length Increased 17%	34%	No	34%
South	In-Plane Shear, El. (-)20'	Embedment Length/Splice Length Increased 17%	18%	No	18%

H.6 Methods Employed to Gain Additional Margin

No recovery of analysis conservatism was required.

Table H-2. Review Results
Electrical Tunnel 'B', Room EF101

Location: 'B' Electrical Tunnel - EF101
Areas Affected by ASR: North and South Walls and Floor Slab El -20'
Calculations: EF-4, Rev. 9 and EF-11, Rev. 1
Drawings: 101610, 101612, 101662, 101664, 101665, 101661

	As Documented																	Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	ν	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
EF-11	South	Vertical Reinforcement El. (-) 20' to 0'	10	Computer Run, One Way Slab, Hand Calc	dead + live + hydro + earth pressure + dynamic earth + OBE	3000	B	B	$A_s = 2$ in^2/ft, E.F.	$A_s = 1.32$ in^2/ft E.F, (E)	34%	Yes	$A_s = 1.32$ in^2/ft, E.F, (E)	34%	EL -20'	Embedment/ Splice Length	No, H	$A_s = 2$ in^2/ft E.F.			No
EF-11	South	In Plane Shear EL. (-) 20' (Horizontal Reinforcement)	10	Computer Run, Hand Calc	Note A	3000	B	B	$A_s = 0.88$ in^2/ft, (C)	$A_{s(min)} = 0.72$ in^2/ft, (C) $A_{s\,req'd} =$ 0.38 in^2 / ft, (C)	18%	Yes	$A_{s(min)} = 0.72$ in^2/ft, (C)	18%	N/A	Embedment/ Splice Length	No, H	$A_s = 0.88$ in^2/ft, (C)			No

Options to Increase Margin:

1. Not used.

Specific Notes:

A. Computer runs were not documented in the calculation, but from other data it appears that the evaluated loads are dead load, live load, hydrostatic, lateral earth pressure, dynamic earth pressure, and seismic.
B. Since computer runs were used to get loads and were not documented, the material properties that were used in the models are unknown. Only material properties used in the hand calculation portion are provided.
C. Reinforcement listed is total from both faces.
D. Not Used.
E. A technical issue with the derivation of this rebar requirement was identified during review and has been communicated to the plant. The technical issue is addressed outside of this review. The margin presented is based on the methodology documented in EF-11.

F. Not used.
G. Not used.
H. Margin for reinforcement required is greater than screening criteria of 17% for development and splice length.

General Notes:

I. The evaluation of the South wall in calculation EF-4 was superseded by the evaluation in calculation EF-11. Thus, only the results of the south wall evaluation in EF-11 are presented.
II. The calculation for the North wall was already reviewed and the results were previously presented in Appendix B, so only the results of the South wall evaluation are presented herein.

I

MS/FW Pipe Chase (East), Exterior

I.1 ASR Affected Areas

The East wall in the Main Steam Feedwater Pipe Chase has pattern cracking along with larger cracks running at various angles on the exterior surface. The area evaluated for potential degradation by ASR is the East wall, above grade elevation (El. 20').

I.2 Reviewed Calculations

Calculation EM-19, Revision 7 was reviewed to determine the documented margin for the East wall of the MS/FW Pipe Chase. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table I-2.

I.3 Calculation General Methodology

Calculation EM-19, Revision 7 evaluated the slabs, columns, beams, and walls above and below grade in the East MS/FW Pipe Chase. The Pipe Chase behaves as a shear wall structure for North-South excitation and as a portal frame for East-West excitation. The vertical and horizontal reinforcement was sized to resist all of the applicable loads, though the horizontal reinforcement was often limited by the minimum reinforcement required by the code.

I.4 Evaluations without Sufficient Anticipated Margin

None.

I.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table I-1 satisfied the screening criteria described in Section 4.0. In some cases, the margin documented in the calculation did not meet the screening criteria, so additional margin was extracted from the evaluation by one of the methods described in Section 5.2. The additional margin was sufficient to satisfy the screening criteria. The applicable evaluations for which additional margin was calculated are indicated in the following table and Section I.6 describes the methods used to extract the additional margin. For the cases in which the documented margin met the screening criteria, the "Anticipated Margin" column reports the documented margin.

Table I-1. Evaluations With Sufficient Anticipated Margin MS/FW Pipe Chase (East), Exterior

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
East: 2' Wall	Out-of-plane Shear	N/A	-0.5%	No	-0.5%
East: 2' Wall	Flexure (vert. rebar)	Embedment & Splice Length Increased 17%	33%	No	33%
East: 2' Wall	Flexure (horiz. rebar)	Embedment & Splice Length Increased 17%	77%	No	77%
East: 2' Wall	Flexure and Axial Load	Embedment & Splice Length Increased 17%	21%	No	21%
East: 2' Wall	Flexure (horiz. rebar)	Embedment & Splice Length Increased 17%	6.4%	Yes	28%
East: 2' Wall	Flexure (horiz. rebar)	Embedment & Splice Length Increased 17%	-0.6%	Yes	23%
East @ El. 22	In-plane Shear	Embedment & Splice Length Increased 17%	77%	No	77%

I.6 Methods Employed to Gain Additional Margin

The splice length reduction factor for staggered splices meeting the requirements for Class B splices was used to identify additional margin for the two evaluations for horizontal rebar subject to pipe whip loads.

Table I-2. Review Results
MS/FW Pipe Chase (East), Exterior

Location: MS/FW Pipe Chase (East)
Areas Affected by ASR: East Wall - Above Grade, Exterior
Calculations: EM-19, Rev. 7
Drawings: 101650, 101652, 101653, FP15781

As Documented																Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Concern?
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment			Actual Rebar Area Present	Other Options	Anticipated Margin	
EM-19	East: 2' Wall	Out-of-plane Shear	95	Computer Run, Hand	Load Comb. 5 (Sh. 63-65)	3000	F	F	V_c = 26.29 k/ft	V_u = 26.42 k/ft	-0.5%	No	0 in^2, (G)	--	N/A	N/A	No	--	--	--	No
EM-19	East: 2' Wall	Flexure (vert. rebar)	96-98	Computer Run, Hand	Load Comb. 5 (Sh 63-65)	3000	F	F	M_c = 270 k-ft	M_u = 187 k-ft	31%, (I)	Yes	$A_{s(req)}$ = 2.09 in^2/ft E.F. (H)	33%	El. 27.3125'	Development/ Splice Length	No, J	A_s = 3.12 in^2 / ft E.F.	--	--	No
EM-19	East: 2' Wall	Flexure (horiz. rebar)	99-101	Computer Run, Hand	Load Comb. 5 (Sh. 63-65)	3000	F	F	M_c = 920 k-ft (C)	M_u = 220 k-ft	76%	Yes	$A_{s(min)}$ = 0.36 in^2 / ft E. F. (N, O)	77%	Edge near E-W Tie Beams	Development/ Splice Length	No, J	A_s = 1.56 in^2 / ft E.F.	--	--	No
EM-19	East: 2' Wall	Flexure and Axial Load	221F-221H	Computer Run, Hand	Pipe Whip Load	3000	F	F	A_s = 1 56 in^2/ ft	A_s = 1.24 in^2/ ft	20.5%	Yes	A_s = 1.24 in^2 / ft E.F.	21%	At North or South Wall	Development/ Splice Length	No, J	A_s = 1.56 in^2 / ft E.F.	--	--	No
EM-19	East: 2' Wall	Flexure (horiz. rebar)	222T, (K)	Computer Run, Hand	Pipe Whip Load	3000	F	F	M_c = 4697 k-ft	M_u = 4416 k-ft	6.0%	Yes	A_s = 1.46 in^2 / ft E.F., (L)	6.4%,	--	Development/ Splice Length	Yes	A_s = 1.56 in^2 / ft E.F.	1	28%	No
EM-19	East. 2' Wall	Flexure (horiz. rebar)	222T, (K)	Computer Run, Hand	Pipe Whip Load	3000	F	F	M_c = 2884 k-ft	M_u = 2910 k-ft (M)	-1%	Yes	A_s = 1.57 in^2 / ft E.F., (L)	-0.6%	--	Development/ Splice Length	Yes	A_s = 1.56 in^2 / ft E.F.	1	23%	No
EM-19	East @ El. 22	In-plane Shear	227	Hand Calc	1.0Ess + 1.0Ra	3000	F	F	V_s = 13473 k V_c = 2704 k	V_u = 3124 k	77% (based on V_s)	Yes	$A_{s(min)}$ = 0.72 in^2 / ft (E, O)	77%	--	Development/ Splice Length	No, J	A_s = 3.12 in^2 / ft	--	--	No

Options to Increase Margin:

1. Credit reduced lap splice requirement for having staggered splices.

Calculation No.	Prepared By	Checked By	Page: I-4
0326-0058-63	Ryan Mi...l	Kevin G...	Revision: 1

Specific Notes:

A. Not used.
B. Not used.
C. A technical issue with the calculation of the flexure capacity was identified during review and has been communicated to the plant. The technical issue is addressed outside of this review. The margin presented is based on the methodology documented in calculation EM-19.
D. Not used.
E. The area of reinforcement required to carry the shear is negligible, minimum reinforcement is controlling.
F. A computer run was used to calculate forces and moments on a frame. The material properties used in the model were not listed.
G. Since the shear capacity was nearly equal the shear demand, no additional rebar was needed despite small negative margin. However, a 25% increase in shear capacity compared to what was documented can be justified (see Section 4.0), resulting in approximately 19.6% margin.
H. Calculated following the same methodology used in the calculation to determine the ultimate moment from a given reinforcing steel area.
I. Loads on West wall are used in the evaluation because they are higher than the loads on the East wall - Sheet 94
J. Margin for reinforcement required is greater than screening criteria of 17% for development and splice length.
K. 222S-222W has the revised (and final) loads and design.
L. The area of reinforcement required was not presented in the calculation. The equations on pages 222E and 222F were used to back-calculate the numbers presented here.
M. The East wall at section b-b was never re-evaluated. The result shows negative margin
N. The minimum reinforcement requirement is more limiting than the steel required to carry the design moment.
O. The minimum reinforcement required is not presented in the calculation. The minimum reinforcement requirement presented herein is based on a reinforcement area to gross section area of 0.0025.
P. Not used.

General Notes:

I. Sheet 3 - f'c=3000psi
II. Not used.
III. Sheet 66 - slight mistake in computer runs, but if the results were not more than 2% different between the incorrect and correct computer run the analysis was not revised
IV. All loads resisted by vertical rebar. Horizontal rebar designed to min code requirements.
V. Conservatively uses west chase pipe break loads - Sheet 48
VI. Sheet 49 - actual accident pressure on wall (Pa) is smaller than what was used in evaluation

J

Cooling Tower, Unit 1 Exterior

J.1 ASR Affected Areas

The ASR affected areas are the South Wall and the North Pipe Chase bump out on the exterior of the building at elevations above grade (above elevation 20 ft.).

J.2 Reviewed Calculations

Calculations CT-53 and CT-28 were reviewed. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table J-3.

J.3 Calculation General Methodology

The analysis is based on a finite element model calculation of stresses in the South wall. Hand calculations are used to combine stress results and an interaction diagram is used for the reinforcement evaluation at some locations.

J.4 Evaluations without Sufficient Anticipated Margin

The evaluations in Table J-1 did not meet the screening criteria described in Section 4.0. In one case, additional margin was calculated above what was documented in the calculation, but the anticipated margin still did not meet the lap splice/embedment length screening criteria of Section 4.0. The applicable evaluation for which additional margin was calculated is indicated in Table J-1 and Section J.6 describes the method used to extract the additional margin. In the other cases, no simplified methods for extracting additional margin were able to be employed. For these cases, the "Anticipated Margin" column reports the documented margin.

Table J-1. Evaluations Without Sufficient Anticipated Margin
Cooling Tower, Unit 1 Exterior

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
South, El. 32 to 39', Cols. A-D	Horiz. reinf. for in-plane shear, bending, and tension	Embedment & Splice Length Increased 17%	2%	Yes	14%
South, El. -8 to 21'	Out of plane shear	Concrete Shear Capacity Reduced 25%	7%	No	7%
South, El. 21 to 45'	Vert. reinf. for bending	Embedment & Splice Length Increased 17%	-2.5%	No	-2.5%
South, El. >50', Cols. D-K	Vert reinf. for bending and tension	Embedment & Splice Length Increased 17%	p. / % 34 / 5.0 35 / 6.3 35 / 8.9 36 / 8.9 37 / 8.9 38 / 11	No	p. / % 34 / 5.0 35 / 6.3 35 / 8.9 36 / 8.9 37 / 8.9 38 / 11

J.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table J-2 satisfied the screening criteria described in Section 4.0. In some cases, the margin documented in the calculation did not meet the screening criteria, so additional margin was extracted from the evaluation by one of the methods described in Section 5.2. The additional margin was sufficient to satisfy the screening criteria. The applicable evaluations for which additional margin was calculated are indicated in Table J-2 and Section J.6 describes the methods used to extract the additional margin. For the cases in which the documented margin met the screening criteria, the "Anticipated Margin" column reports the documented margin.

Table J-2. Evaluations With Sufficient Anticipated Margin
Cooling Tower, Unit 1 Exterior

<table>
<tr><th>Wall</th><th>Evaluation</th><th>ASR Effect</th><th>Documented Margin (%)</th><th>Additional Margin Calculated?</th><th>Anticipated Margin (%)</th></tr>
<tr><td>South, El. -8 to 45', Cols. A-D</td><td>Horiz reinf. for in-plane shear, bending, and tension</td><td>Embedment & Splice Length Increased 17%</td><td><table><tr><td>El.</td><td>%</td></tr><tr><td>-8 to 12</td><td>51</td></tr><tr><td>12 to 22</td><td>26</td></tr><tr><td>22 to 25</td><td>8</td></tr><tr><td>25 to 32</td><td>8</td></tr><tr><td>39 to 45</td><td>10</td></tr></table></td><td>Yes</td><td><table><tr><td>El.</td><td>%</td></tr><tr><td>-8 to 12</td><td>51</td></tr><tr><td>12 to 22</td><td>26</td></tr><tr><td>22 to 25</td><td>29</td></tr><tr><td>25 to 32</td><td>29</td></tr><tr><td>39 to 45</td><td>31</td></tr></table></td></tr>
<tr><td>South, El. -8 to 38', Cols. D-G</td><td>Horiz reinf. for in-plane shear, bending, and tension</td><td>Embedment & Splice Length Increased 17%</td><td><table><tr><td>El.</td><td>%</td></tr><tr><td>-8 to 12</td><td>53</td></tr><tr><td>12 to 22</td><td>20</td></tr><tr><td>22 to 25</td><td>12</td></tr><tr><td>25 to 32</td><td>14</td></tr><tr><td>32 to 38</td><td>-0.6</td></tr></table></td><td>Yes</td><td><table><tr><td>El.</td><td>%</td></tr><tr><td>-8 to 12</td><td>53</td></tr><tr><td>12 to 22</td><td>20</td></tr><tr><td>22 to 25</td><td>33</td></tr><tr><td>25 to 32</td><td>34</td></tr><tr><td>32 to 38</td><td>23</td></tr></table></td></tr>
<tr><td>South, El. 21 to 45'</td><td>Out of Plane Shear</td><td>NA</td><td>5%</td><td>No</td><td>5%</td></tr>
<tr><td>South, El. -8 to 21', Cols. A-D</td><td>Vert. reinf. for bending</td><td>Embedment & Splice Length Increased 17%</td><td>25%</td><td>No</td><td>25%</td></tr>
<tr><td>South, El. -8 to 21', Cols. D-G</td><td>Vert. reinf. for bending</td><td>Embedment & Splice Length Increased 17%</td><td>50%</td><td>No</td><td>50%</td></tr>
<tr><td>South, El. 45'-77', Cols. A-D, K-N</td><td>Horiz reinf. for in-plane shear, bending, and tension</td><td>Embedment & Splice Length Increased 17%</td><td><table><tr><td>El.</td><td>%</td></tr><tr><td>45 to 53</td><td>45</td></tr><tr><td>53 to 61</td><td>41</td></tr><tr><td>61 to 69</td><td>53</td></tr><tr><td>69 to 77</td><td>53</td></tr></table></td><td>No</td><td><table><tr><td>El.</td><td>%</td></tr><tr><td>45 to 53</td><td>45</td></tr><tr><td>53 to 61</td><td>41</td></tr><tr><td>61 to 69</td><td>53</td></tr><tr><td>69 to 77</td><td>53</td></tr></table></td></tr>
</table>

Table J-2. Evaluations With Sufficient Anticipated Margin
Cooling Tower, Unit 1 Exterior

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
South, El. 46-76', Cols. A-D, K-N	Vert. reinf. for bending and tension	Embedment & Splice Length Increased 17%	33%	No	33%
South, El. 46-76', Cols. A-D, K-N	Vert. reinf. for bending and max comp.	Embedment & Splice Length Increased 17%	77%	No	77%
South, El. 46-76', Cols. A-D, K-N	Vert. reinf. for max bending and comp.	Embedment & Splice Length Increased 17%	33%	No	33%
South, El. 46-76', Cols. A-D, K-N	Out of Plane Shear	NA	89%	No	89%
South, El. >50', Cols. D-K	Horiz reinf. for in-plane shear, bending, and tension	Embedment & Splice Length Increased 17%	p. % 26 3.4 27 1.0 29 4.8 31 33 31 37 32 19 33 18	Yes	p. % 26 26 27 24 29 27 31 33 31 37 32 19 33 18
South, El. >50', Cols. D-K	Out of Plane Shear	NA	p. % 40 4.1 42 -3.0	No	p. % 40 4.1 42 -3.0
South, El. 50-77', Cols. F and H	Horiz. reinf for bending, tension, and shear	Embedment & Splice Length Increased 17%	27%	No	27%

Table J-2. Evaluations With Sufficient Anticipated Margin
Cooling Tower, Unit 1 Exterior

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
South, El. 50', Cols. D-F, H-K	In plane shear	NA	Significant[10]	No	Significant[10]
South, El. 50', Cols. D-F, H-K	Out of Plane Shear	NA	0%	No	0%
South, El. 50-59', Cols. D & F	Horiz. reinf for bending, tension, and shear	Embedment & Splice Length Increased 17%	0%	Yes	24%
South, El. 50', Cols. D-K	Out of Plane Shear	NA	7.3%	No	7.3%

J.6 Methods Employed to Gain Additional Margin

The following method was used to recover margin from the calculation:

- Where lap splices are staggered, i.e., adjacent rows of reinforcement do not have the overlap at the same axial location, these are Class B lap splices (ACI 318, 7.6.3.1.1). The development length requirement for a Class B splice is $1.3l_d$. The actual development length used in the construction is $1.7l_d$, which is the development length requirement for a Class C splice (see ACI 318, 7.6.1.3 for ACI Class C splice and Reference 2 for Seabrook drawing requirement for lap splice length.). Accordingly, the margin in the development length is (1.7-1.3)/1.7=23.5%.

[10] CT-28 did not calculate the margin, but concluded the margin was significant. Approximating the shear strength of unreinforced concrete as $2\sqrt{f'_c}$=126 psi, the margin would be 52%.

Table J-3. Review Results
Cooling Tower, Unit 1 Exterior

Location: Cooling Tower
Areas Affected by ASR: South Wall and North Pipe Chase Bump Out - Exterior Above Grade
Calculations: CT-53, Rev. 1 and CT-28, Rev. 6
Drawings: 101701, 101702, 101707, 101708, 101709, 101710, 101716, 101717, 101718, 101719, FP12183, FP12186, FP12344, FP13714, FP13715

As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credit?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Loc of Max Mom	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Conc.?
CT-53	South, El. -8 to 45', Cols. A-D	Horiz reinf. for in-plane shear, bending, and tension	37	Finite element model, Hand Calc	Hydro, soil, seismic, equip.	4000	E	E	El. | in^2/ft/ef -8 to 12 | 2.08 12 to 22 | 2.08 22 to 25 | 1.56 25 to 32 | 1.56 32 to 39 | 1.56 39 to 45 | 1.56	El. | in^2/ft/ef -8 to 12 | 1.01 12 to 22 | 1.54 22 to 25 | 1.44 25 to 32 | 1.44 32 to 39 | 1.53 39 to 45 | 1 40	El. | % -8 to 12 | 51 12 to 22 | 26 22 to 25 | 8 25 to 32 | 8 32 to 39 | 2 39 to 45 | 10 Note A.	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	Yes	Same as Cap. column	1, 2, 3 Dwgs. FP-12344, FP-12186, FP-12183	El. | % -8 to 12 | NA 12 to 22 | NA 22 to 25 | 29 25 to 32 | 29 32 to 39 | 14 39 to 45 | 31	Yes
CT-53	South, El. -8 to 38', Cols. D-G	Horiz reinf. for in-plane shear, bending, and tension	37	Finite element model, Hand Calc	Hydro, soil, seismic, equip.	4000	E	E	El. | in^2/ft/ef -8 to 12 | 2.08 12 to 22 | 2.08 22 to 25 | 1.56 25 to 32 | 1.56 32 to 38 | 1.56	El. | in^2/ft/ef -8 to 12 | 0.97 12 to 22 | 1.67 22 to 25 | 1.37 25 to 32 | 1.34 32 to 38 | 1.57	El. | % -8 to 12 | 53 12 to 22 | 20 22 to 25 | 12 25 to 32 | 14 32 to 38 | -0.6 Note B.	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	Yes	Same as Cap. column	1, 2 Dwgs. FP-12344, FP-12186, FP-12183	El. | % -8 to 12 | NA 12 to 22 | NA 22 to 25 | 33 25 to 32 | 34 32 to 38 | 23	No
CT-53	South, El. -8 to 21'	Out of plane shear	38	FEA, Hand Calc	Hydro, soil, seismic, equip	4000	E	E	111 psi	103, Note F	7%	No				Out of plane shear	Yes		1	7%	Yes
CT-53	South, El. 21 to 45'	Out of plane shear	39	FEA, Hand Calc	Hydro, seismic, equip	4000	E	E	102 psi	97	5%	No				NA	No				No
CT-53	South, El. -8 to 21', Cols. A-D	Vert. reinf. for bending	41	FEA, Hand Calc	Hydro, soil, seismic, equip.	4000	E	E	2.08 in^2/ft/EF	1.55 in^2/ft/EF	25%	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	No	Same as Cap. column			No

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As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credit?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Loc of Max Mom	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Conc.?
CT-53	South, El. -8 to 21', Cols. D-G	Vert. reinf. for bending	40	FEA, Hand Calc	Hydro, soil, seismic, equip.	4000	E	E	3.12 in^2/ft/EF	1.55 in^2/ft/EF	50%	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	No	Same as Cap. column			No
CT-53	South, El. 21 to 45'	Vert. reinf. for bending	41	FEA, Hand Calc	Hydro, seismic, equip.	4000	E	E	0.79 in^2/ft/EF	0.81 in^2/ft/EF	-2.5% Note C	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	Yes	Same as Cap. column	1	-2.5%	Yes
CT-28	South, El. 45'-77', Cols. A-D, K-N	Horiz reinf. for in-plane shear, bending, and tension	25	FEA, Hand Calc	Hydro, seismic, equip.	4000	E	E	El. | in^2/ft/ef 45 to 53 | 1.33 53 to 61 | 1.33 61 to 69 | 1.33 69 to 77 | 1.33	El. | in^2/ft/ef 45 to 53 | 0.73 53 to 61 | 0.79 61 to 69 | 0.62 69 to 77 | 0.62	El | % 45 to 53 | 45 53 to 61 | 41 61 to 69 | 53 69 to 77 | 53	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	No	Same as Cap. column			No
CT-28	South, El. 46-76', Cols. A-D, K-N	Vert. reinf. for bending and tension	22	FEA, Hand Calc	Hydro, seismic, equip.	4000	E	E	0.79 in^2/ft/EF	0.53 in^2/ft/EF	33%	Yes	0.53 in^2/ft/EF	33%		Splice Lap Length	No	0.79 in^2/ft/EF			No
CT-28	South, El. 46-76', Cols. A-D, K-N	Vert. reinf. for bending and max comp.	23	FEA, Interaction Diag.	Hydro, seismic, equip.	4000	E	E	125 ft*K (p. 13) Note G	P_u = -93.9 K M_u = 28.4 ft*K	77%	Yes	0.17 in^2/ft/EF Note H	77%		Splice Lap Length	No	0.79 in^2/ft/EF			No
CT-28	South, El. 46-76', Cols. A-D, K-N	Vert. reinf. for max bending and comp.	23	FEA, Interaction Diag.	Hydro, seismic, equip.	4000	E	E	90 ft*K (p. 13) Note G	P_u = -35.0 K M_u = 60.2 ft*K	33% Note D	Yes	0.33 in^2/ft/EF	33% Note D		Splice Lap Length	No	0.79 in^2/ft/EF			No
CT-28	South, El. 46-76', Cols. A-D, K-N	Transverse shear	24	FEA, Hand Calc	Hydro, seismic, equip.	4000	E	E	126 psi	14.3 psi	89%	No				NA	No				No

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As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	ν	Capacity	Demand	Margin	Rebar Credit?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Loc of Max Mom	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Conc.?
CT-28	South, El. >50', Cols. D-K	Horiz reinf. for in-plane shear, bending, and tension	26, 27, 29, 31, 32, 33	FEA, Hand Calc	Hydro, seismic, equip.	4000	E	E	2.08 in²/ft/EF	p. | in²/ft/ef 26 | 2.01 27 | 2.06 29 | 1.98 31 | 1.4 31 | 1.32 32 | 1.69 33 | 1.7	p. | % 26 | 3.4 27 | 1.0 29 | 4.8 31 | 33 31 | 37 32 | 19 33 | 18	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	Yes	Same as Cap. column	1, 2 Dwgs. FP-13714, FP-13715	p. | % 26 | 26 27 | 24 29 | 27 31 | NA 31 | NA 32 | NA 33 | NA	No
CT-28	South, El. >50', Cols. D-K	Vert reinf. for bending and tension	34-38	FEA, Hand Calc	Hydro, seismic, equip.	4000	E	E	0.79 in²/ft/EF	p. | in²/ft/ef 34 | 0.75 35 | 0.74 35 | 0.72 36 | 0.72 37 | 0.72 38 | 0.70	p. | % 34 | 5.0 35 | 6.3 35 | 8.9 36 | 8.9 37 | 8.9 38 | 11	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	Yes	Same as Cap. column	1	p. | % 34 | 5.0 35 | 6.3 35 | 8.9 36 | 8.9 37 | 8.9 38 | 11	Yes
CT-28	South, El. >50', Cols. D-K	Transverse shear	40, 42	FEA, Hand Calc	Hydro, seismic, equip.	4000	E	E	p. | psi 40 | 65.3 42 | 91.0	p. | psi 40 | 62.6 42 | 93.8	p. | % 40 | 4.1 42 | -3.0	No				NA	No				No
CT-28	South, El. 50-77', Cols. F and H	Horiz. reinf for bending, tension, and shear	55	FEA, Hand Calc	Hydro, seismic, equip., Add'l load from column	4000	E	E	2.08 in²/ft/EF	1.52 in²/ft/EF	27%	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	No	Same as Cap. column			No
CT-28	South, El. 50', Cols D-F, H-K	In plane shear	56	FEA, Hand Calc	Hydro, seismic, equip., Add'l load from column	4000	E	E	>>60 psi	60 psi	Significant (Note 1)	No				NA	No				No

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As Documented																Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Options to Increase Margin			ASR Conc.?
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E	v	Capacity	Demand	Margin	Rebar Credit?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Loc of Max Mom			Actual Rebar Area Present	Other Options	Anticipated Margin	
CT-28	South, El. 50', Cols. D-F, H-K	Transverse shear	59	FEA, Hand Calc	Hydro, seismic, equip., Add'l load from column	4775	E	E	107.8 psi	107.8 psi	0%	No				NA	No				No
CT-28	South, El. 50-59', Cols. D & F	Horiz. reinf for bending, tension, and shear	65	FEA, Hand Calc	Hydro, seismic, equip., Add'l load from column	4000	E	E	2.08 in^2/ft/EF	2.08 in^2/ft/EF	0%	Yes	Same as Demand column	Same as Margin column		Splice Lap Length	Yes	Same as Cap. column	1, 2	24	No
CT-28	South, El. 50', Cols. D-K	Transverse shear	67	FEA, Hand Calc	Hydro, seismic, equip., Add'l load from column	4000	E	E	86.8 psi	80.5 psi	7.3%	No				NA	No				No

Options to Increase Margin:

1. The analysis is based on a detailed finite element model. The results show that many locations have small margins. The analysis has locations that have negative margin in the first analysis pass and refinements of the analysis are used to get an acceptable result. It does not appear there will be margin gain from a finite element analysis or from refinements in the application of the ACI code.
2. The lap splices at this location are staggered, i.e., adjacent rows of reinforcement do not have the overlap at the same axial location. Every other row of reinforcement is at the same axial location. These are Class B lap splices (ACI 318, 7.6.3.1.1). The development length requirement for a Class B splice is $1.3l_d$. The actual development length used in the construction is $1.7l_d$, which is the development length requirement for a Class C splice (see ACI 318, 7.6.1.3 for ACI Class C splice and Reference 2 for Seabrook drawing requirement for lap splice length.). Accordingly, the margin in the development length is (1.7-1.3)/1.7=23.5%.
3. The required rebar for this location of 1.53 in^2/ft/EF is based on an average from Elev. 25.33 to 45.0 (p. 37 of CT-53). The analysis to recover margin did not use this averaging approach because it is considered more accurate to average over a length of approximately three times the wall thickness, or about the size of one element in the CT-53 finite element model.

Specific Notes:

A. Deleted.
B. The negative margin at Elev. 32 to 38 feet is not addressed Calculation CT-53.
C. Page 41 of the calculation states that the supplied rebar is close enough to the required rebar to be OK.
D. The margin may be less than this based on Rev. 4 and the modification of the phi factor as stated in the note on p. 23.

E. A finite element analysis was used to calculate loads on the south wall. The elastic modulus and Poisson's ratio used for concrete are not listed in the document.
F. There is a higher demand at Elevations -8' to 4', however, the shear is in a direction such that the wall bears against fill concrete and the shear load will transfer directly to this medium (see p. 38). Since this location is below the elevation of concern for ASR (above grade elevation 20'), the shear at this location is not presented in the summary table.
G. The capacity was calculated by determining the maximum allowable moment for #8 rebar @12" given an axial load equal to P_c as provided in the Demand column.
H. The required reinforcement was estimated based on the percentage margin provided in the Margin column. Given the location of the point on the interaction diagram, this is likely conservative.
I. The amount of margin is not calculated in CT-28, but assuming a concrete allowable shear stress of $2\sqrt{f'_c}$=126 psi, the margin would be 52%.

General Notes:

1. The calculation is an explicit analysis of the South wall, which is also applicable to the North wall due to the similarity of the walls. There is no evaluation for the pipe chase bump out on the North wall.

K

Service Water Pumphouse, Exterior

K.1 ASR Affected Areas

The North wall of the Southwest bump out and the South wall of the Service Water Pumphouse have intermittent, localized pattern cracking on the exterior surface. The areas evaluated for potential degradation by ASR are the North wall of the Southwest bump out and the South wall, above grade elevation (El. 20').

K.2 Reviewed Calculations

Calculation CW-29, Revision 7 was reviewed to determine the documented margin for the North wall of the Southwest bump out and the South wall of the Service Water Pumphouse. Only the relevant information pertaining to the evaluation of the ASR-affected areas listed above was reviewed as part of this effort. The results of the review are presented in Table K-2.

In addition, calculation SBSAG-1MA, Revision 1 was reviewed to determine the design basis for the tornado missile loads for the Service Water Pumphouse.

K.3 Calculation General Methodology

Calculation CW-29, Revision 7 evaluated the walls above grade, the roof slab, and the hatch covers in the Service Water Pumphouse. The design of the walls was based on a computer program, which output loads on different sections of the wall. Regardless of the calculation, No. 8 bars at 12" spacing were provided in the exterior walls to meet the missile shield requirements. Based on the loads documented in calculation CW-29, the loads on the exterior walls generally required less reinforcement than the provided amount.

Calculation SBSAG-1MA, Revision 1 evaluated the tornado missile loads for the Service Water Pumphouse. The calculation compared the capacity of the exterior walls to the demand from a steel pipe, automobile and wood pole tornado missile. The calculation also evaluated a typical wall for light aircraft impact loads and calculated the impact velocities at which failure was anticipated.

K.4 Evaluations without Sufficient Anticipated Margin

None.

K.5 Evaluations with Sufficient Anticipated Margin

The evaluations in Table K-1 satisfied the screening criteria described in Section 4.0. For all cases, the "Anticipated Margin" column reports the documented margin. Note that the minimum reinforcement to react tornado missile loads was not calculated in SBSAG-1MA. To calculate the margin for the tornado missile evaluation, a scoping analysis was performed following the same methodology documented in SBSAG-1MA, but with progressively lower values of reinforcement area. Note that the light-aircraft impact analysis was not re-evaluated, since that portion of the calculation was for a "typical" wall and did not represent the limiting wall in the structure. The results for a "typical" wall are unchanged.

Table K-1. Evaluations With Sufficient Anticipated Margin Service Water Pumphouse, Exterior

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
Wall 10 -North Wall	Flexure Vertical	Embedment & Splice Length Increased 17%	44%	No	44%
Wall 10 -North Wall	Flexure Horiz.	Embedment & Splice Length Increased 17%	54%	No	54%
Wall 10 -North Wall	In-Plane Shear	Embedment & Splice Length Increased 17%	54%	No	54%
Wall 13 - South Wall	Flexure Vertical	Embedment & Splice Length Increased 17%	42%	No	42%
Wall 13 - South Wall	Flexure Horiz.	Embedment & Splice Length Increased 17%	54%	No	54%
Wall 13 - South Wall	In-Plane Shear	Embedment & Splice Length Increased 17%	54%	No	54%
Wall 10 -North Wall	Core Bores: Vertical Reinf.	N/A	21%	No	21%
Wall 10 -North Wall	Core Bores: Horiz. Reinf.	Embedment & Splice Length Increased 17%	41%	No	41%

Table K-1. Evaluations With Sufficient Anticipated Margin
Service Water Pumphouse, Exterior

Wall	Evaluation	ASR Effect	Documented Margin (%)	Additional Margin Calculated?	Anticipated Margin (%)
Limiting Wall of Service Water Pumphouse	Response of 12" dia. x 15' Steel Pipe Missile	Embedment & Splice Length Increased 17%	44%	No	44%
Limiting Wall of Service Water Pumphouse	Response of Automobile Missile	Embedment & Splice Length Increased 17%	44%	No	44%
Limiting Wall of Service Water Pumphouse	Response of Wood Pole Missile	Embedment & Splice Length Increased 17%	44%	No	44%

K.6 Methods Employed to Gain Additional Margin

The documented margin for the evaluations of the North and South walls met the screening criteria of Section 4.0. No additional margin needed to be calculated.

Table K-2. Review Results
Service Water Pumphouse, Exterior

Location:	Service Water Pumphouse
Areas Affected by ASR:	North Wall of SW Bump out & South Wall - Above Grade, Exterior
Calculations:	CW-29, Rev. 7 and SBSAG-1MA, Rev.1 (Tornado Missile Load Evaluation)
Drawings:	101086, 101094, 101093

As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E (psi)	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{s(Req'd)}}{A_{s(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
CW-29	Wall 10 - North Wall	Flexure Vertical	37-38	FEA, Interaction Diagram	Load Eq. 7, (N)	4000	F	F	$A_s = 0.79$ in^2/ft E. F. (A)	$A_s = 0.44$ in^2/ft E. F. (H)	44%	Yes	$A_s = 0.44$ in^2/ft E. F. (H)	44%	(O)	Development/ Splice Length	No	$A_s = 0.79$ in^2 /ft E. F.	--	--	No
CW-29	Wall 10 - North Wall	Flexure Horiz.	39-40	FEA, Interaction Diagram	Load Eq. 15, (N)	4000	F	F	$A_s = 0.79$ in^2/ft E. F., (A)	$A_{s(min)} = 0.36$ in^2 (B)	54%	Yes	$A_{s(min)} =$ 0.36 in^2 (B)	54%	(O)	Development/ Splice Length	No	$A_s = 0.79$ in^2 /ft E. F.	--	--	No
CW-29	Wall 10 - North Wall	In-Plane Shear	41	FEA, Hand Calc	Load Eq. 6, (N)	4000	F	F	$V_c = 634$k $A_s = 1.58$ in^2 / ft	$V_u = 675.3$k $A_{s(min)} =$ 0.72 in^2 / ft (E, I)	54%	Yes	$A_{s(min)} =$ 0.36 in^2 / ft E. F (E)	54%	N/A	Development/ Splice Length	No	$A_s = 0.79$ in^2 /ft E. F.	--	--	No
CW-29	Wall 13 - South Wall	Flexure Vertical	45-46	FEA, ACI SP-17	Load Eq. 5, (N)	4000	F	F	$A_s = 0.79$ in^2 /ft E. F.	$A_s = 0.46$ in^2 / ft E. F. (H)	42%	Yes	$A_s = 0.46$ in^2 / ft E. F. (H)	42%	(O)	Development/ Splice Length	No	$A_s = 0.79$ in^2 /ft E. F.	--	--	No
CW-29	Wall 13 - South Wall	Flexure Horiz.	47-48	FEA, ACI SP-17	Load Eq. 7, (N)	4000	F	F	$A_s = 0.79$ in^2 /ft E. F.	$A_s = 0.31$ in^2/ft E. F. (H, J) $A_{s(min)} =$ 0.36 in^2/ft E. F. (E, G)	54%	Yes	$A_s = 0.31$ in^2/ft E. F. (H, J) $A_{s(min)} =$ 0.36 in^2/ft E. F. (E, G)	54%	(O)	Development/ Splice Length	No	$A_s = 0.79$ in^2 /ft E. F.	--	--	No
CW-29	Wall 13 - South Wall	In-Plane Shear	49-50	FEA, Hand Calc	Load Eq. 8, (N)	4000	F	F	$V_c = 2983$ k $A_s = 0.79$ in^2 /ft E. F.	$V_u = 1499$ k $A_s = 0.31$ in^2/ft E. F. (H, J)	49%, (P)	Yes	$A_s = 0.31$ in^2/ft E. F. (H,J) $A_{s(min)} =$ 0.36 in^2/ft E. F. (E, G)	54%	N/A	Development/ Splice Length	No	$A_s = 0.79$ in^2 /ft E. F.	--	--	No

MPR

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Calculation No.	Prepared By	Checked By	Page: K-5
0326-0058-63	[signature]	[signature]	Revision: 1

As Documented																		Options to Increase Margin			
Calc	Wall	Evaluation	Pages	Method	Loads	f'_c (psi)	E (psi)	v	Capacity	Demand	Margin	Rebar Credited?	Rebar Area Required	$1-\frac{A_{S(Req'd)}}{A_{S(Present)}}$	Location of Max Moment	Structural Concern due to ASR	Consider Options to Reduce ASR Risk?	Actual Rebar Area Present	Other Options	Anticipated Margin	ASR Concern?
CW-29	Wall 10 - North Wall	Core Bores: Vertical Reinf.	M4-M5	Hand Calc	N/A	4000	F	F	Reserve: A_s = 1 197 in^2 Cut: A_s =0.79 in^2 Remaining: A_s = 0.407 in^2. (K)	A_s = 1.505 in^2 (K)	21%, (C)	Yes	A_s = 1.505 in^2 (K)	21%, (C)	N/A	N/A, (L)	No	A_s = 1.912 in^2 One Face	--	--	No
CW-29	Wall 10 - North Wall	Core Bores: Horiz. Reinf.	M4-M5	Hand Calc	N/A	4000	F	F	Reserve: A_s = 1.86 in^2 Cut: A_s = 0.79 in^2 Remaining: A_s = 1.07 in^2 (M)	A_s = 1.56 in^2 (M)	41%, (C)	Yes	A_s = 1.56 in^2 (M)	41%, (C)	N/A	Development/ Splice Length	No	A_s = 2.63 in^2 One Face	--	--	No
SBSAG-M1	Limiting Wall of Service Water Pumphouse	Response of 12" dia. x 15' Steel Pipe Missile	60, 77	SRP 3.5.3	12" dia. x 15' Steel Pipe Missile	3000	3.12e6	0.15	μ'_{all} = 10.3	$\bar{\mu}$ = 1.45	86% (Q)	Yes	A_s =0.44 in^2/ft, E F., (R)	44%	N/A	Development/ Splice Length	No	A_s = 0.79 in^2 /ft E. F.	--	--	No
SBSAG-M1	Limiting Wall of Service Water Pumphouse	Response of Automobile Missile	60, 81	SRP 3.5.3	Auto. Missile	3000	3.12e6	0.15	μ'_{all} = 10.3	$\bar{\mu} \ll 1$	> 90% (Q)	Yes	A_s =0.44 in^2/ft, E. F., (R)	44%	N/A	Development/ Splice Length	No	A_s = 0.79 in^2 /ft E. F.	--	--	No
SBSAG-M1	Limiting Wall of Service Water Pumphouse	Response of Wood Pole Missile	70, 85	SRP 3.5.3	Wood Pole Missile	3000	3.12e6	0.15	μ'_{all} = 1.32	$\bar{\mu} < 1$	>24% (Q)	Yes	A_s =0.44 in^2/ft, E. F., (R)	44%	N/A	Development/ Splice Length	No	A_s = 0.79 in^2 /ft E. F.	--	--	No

General Notes:

1. The walls were designed with #8 @ 12" for tornado missile loads. The methodology of Calculation SBSAG-1MA was followed with progressively lower steel reinforcement areas and it was determined that #6 @ 12" reinforcement is adequate. Note that the light-aircraft impact analysis was not re-evaluated, since that portion of the calculation was for a "typical" wall and did not represent the limiting wall in the structure. The results for a "typical" wall are unchanged.

Specific Notes:

A. The minimum required $A_s = 0.36$ in^2/ft. However, the capacity is based on #8 bars at 12" spacing for satisfying the missile shield requirement.
B. The required steel for horizontal flexure and in-plane shear loads is less than the minimum required steel.
C. Does not include rebar required for reacting a tornado missile.
D. Not used.
E. Based on minimum requirement for walls.
F. Computer model was used to extract loads on walls. It is not known what concrete elastic modulus or Poisson's ratio were used in the model.
G. The minimum reinforcement area was not presented in the calculation for this wall, but it is limiting. Therefore, it is calculated and presented herein.
H. Reinforcement required from design basis loads excluding tornado missile evaluation.
I. Total reinforcement requirement for the full section (sum of both tension and compression faces).
J. Requirement from the combination of both in-plane shear and horizontal flexure reinforcement calculations.
K. Remaining rebar is in excess of that required by analysis over a 3'-5" span. That required by analysis is presented in the "Demand" column (area required per foot times span).
L. There are no splices in the region of the core bore (DWG 101094).
M. Remaining rebar is in excess of that required by analysis over a 4'-4" span. That required by analysis is presented in the "Demand" column (area required per foot times span).
N. The component loads in each load case are not described in this document.
O. Location of the maximum moment was not described in the results.
P. Margin based on shear loads.
Q. Based on reported Demand and Capacity. These factors may not accurately represent the margin present since the parameters are non-linear.
R. The required reinforcement was calculated following the same methodology presented in SBSAG-M1, but with decreasing values of steel for a 17.5' × 23.33' wall (representing the north wall of the bump out). The procedure was stopped at $A_s = 0.44$ in^2/ft and the failure limit had not been reached. The wall may be acceptable with less steel.

L

Implementation of Reinforcement Embedment Criterion

ASR has the potential to reduce the effective strength of reinforced concrete at locations of reinforcement lap splices and at locations of reinforcement straight bar embedment in areas where a three-dimensional reinforcement cage is not provided. Additional length is required in the reinforcement lap splice length and in the embedment length to fully develop the strength of the reinforcing steel in ASR-affected areas.

The sections below calculate the splice and embedment length required in ASR-affected areas for two loading cases: flexure and in-plane-shear. The calculations consider a potential 40% strength reduction in the splice as observed by Chana (Reference 8) and 23% conservatism in the ACI Code equations for reinforcement lap splice strength as documented in ACI Code Committee reports (Reference 5).

L.1 Reinforcement Embedment in Flexure

This section considers the moment capacity of a reinforced concrete section with rebar on the tension face, neglecting any rebar on the compression face. Neglecting rebar on the compression face is a conservative assumption, typically used in design, and is neglected in many of the safety-related calculations for Seabrook Station. The rebar on the compression face is neglected in this evaluation for simplicity and this approach does not alter the final conclusion. The calculation below will determine the effect of the potential 40% strength reduction and the 23% code conservatism on moment capacity.

The moment capacity M_c of a concrete section crediting the rebar area A_s to carry tension and the concrete to react compression (assuming a rectangular concrete stress block; derived following Reference 3, Section 10.3.1) is:

$$M_c = A_s f_y\left(d - \tfrac{a}{2}\right) \qquad (1)$$

where f_y is the specified yield strength of reinforcing steel, d is the distance from the concrete extreme compression fiber to the centerline of reinforcing bars in tension, and a is the depth of the concrete compression block.

The required development length l_d of a reinforcing bar of size #11 or less (applicable to all areas of concern for ASR degradation) (Reference 3, Section 12.5(a)) is:

$$l_d = \frac{0.04 A_b f_y}{\sqrt{f'_c}} \qquad (2)$$ [11]

where A_b is the cross-sectional area of a single bar and f'_c is the specified compressive strength of the concrete.

The required development length of a reinforcing bar accounting for the actual area of steel provided ($A_{s(prov)}$) beyond that required by design ($A_{s(req)}$) (Reference 3, Sec. 12.5(d)) and a potential 40% reduction of development strength due to ASR (Reference 4, Table 4) is:

$$l_{d(ASR)} = 1.4 l_d \frac{A_{s(req)}}{A_{s(prov)}} \qquad (3)$$ [12]

In a case where the required development length is not provided, the allowable stress on the rebar must be reduced. The relationship between the required development length and the allowable rebar stress $f_{y(eff)}$ is:

$$\frac{l_{d(ASR)}}{f_y} = \frac{l_{d(prov)}}{f_{y(eff)}} \qquad (4)$$

Solving the relationship in Eq. 4 for $f_{y(eff)}$ results in:

$$f_{y(eff)} = \frac{l_{d(prov)} f_y}{l_{d(ASR)}} \qquad (5)$$

Substituting $f_{y(eff)}$ for f_y in Eq. 1 and considering a 23% strength increase relative to the ACI Code Equations (as justified in Reference 5) results in:

$$M_c = 1.23 A_{s(prov)} f_{y(eff)} \left(d - \frac{a}{2}\right) \qquad (6)$$

Substituting the right side of Eq. 5 for $f_{y(eff)}$ results in:

$$M_c = 1.23 A_{s(prov)} \left(\frac{l_{d(prov)} f_y}{l_{d(ASR)}}\right) \left(d - \frac{a}{2}\right) \qquad (7)$$

[11] The variable $\sqrt{f'_c}$ carries the units psi and the coefficient carries the units $\frac{1}{in}$.
[12] See Section 5.2 of this calculation for a discussion of the applicability of the factor $A_{s(req)}/A_{s(prov)}$ to reinforcement which requires development for full f_y.

Substituting the right side of Eq. 3 for $l_{d(ASR)}$ results in:

$$M_c = 1.23A_{s(prov)}\left(\frac{l_{d(prov)}f_y}{1.4l_d\frac{A_{s(req)}}{A_{s(prov)}}}\right)\left(d-\frac{a}{2}\right) \tag{8}$$

Simplifying the expression:

$$M_c = \left(\frac{1.23}{1.4}\right)\left(\frac{A_{s(prov)}{}^2}{A_{s(req)}}\right)\left(\frac{l_{d(prov)}f_y}{l_d}\right)\left(d-\frac{a}{2}\right) \tag{9}$$

Substituting the right side of Eq. 2 for l_d and setting A_b equal to $A_{s(prov)}$ results in the moment capacity for a wall width equal to the rebar spacing:

$$M_c = \left(\frac{1.23}{1.4}\right)\left(\frac{A_{s(prov)}{}^2}{A_{s(req)}}\right)\left(\frac{l_{d(prov)}f_y}{\frac{0.04A_{s(prov)}f_y}{\sqrt{f'c}}}\right)\left(d-\frac{a}{2}\right) \tag{10}$$

Simplifying the expression:

$$M_c = (0.879)\left(\frac{A_{s(prov)}}{A_{s(req)}}\right)\left(\frac{l_{d(prov)}\sqrt{f'c}}{0.04}\right)\left(d-\frac{a}{2}\right) \tag{11}$$

The effect of the potential strength reduction and the documented code conservatism on the moment capacity is a factor of 0.879 on capacity, or a 13.8% reduction in margin. The 13.8% margin reduction is less than the 17% margin requirement that results from the arithmetic sum of the 40% reduction and the 23% increase. Therefore, it is conservative to use 17% as the screening criterion.[13]

L.2 Reinforcement Embedment in In-Plane-Shear

This section considers the in-plane shear capacity of a reinforced concrete section that credits rebar in shear. The shear capacity of a reinforced concrete section is composed of two terms. One term represents the shear capacity of the plain concrete. The second term represents the additional shear capacity provided by the rebar. The calculation below will determine the effect of the potential 40% strength reduction and the 23% code conservatism.

The shear capacity V_s provided by the rebar (Reference 3, Equation 11-13, where $v_s = v_u - v_c$) is:

[13] It is noted that the calculated 13.8% reduction in margin is also a conservative result. The actual reduction in margin is less than 13.8% because the depth of the concrete compression block could be recalculated for the reduced effective reinforcement strength.

$$v_s = \frac{A_v f_y}{b_w s} \tag{12}$$

where A_v is the area of shear reinforcement, f_y is the reinforcement specified yield strength, b_w is the thickness of the wall, and s is the shear reinforcement spacing.

The required development length l_d of a reinforcing bar of size #11 or less (applicable to all areas of concern for ASR degradation) (Reference 3, Section 12.5(a)) is:

$$l_d = \frac{0.04 A_b f_y}{\sqrt{f'_c}}^{14} \tag{13}$$

where A_b is the cross-sectional area of a single bar and f'_c is the specified compressive strength of the concrete.

The required development length of a reinforcing bar accounting for the actual area of steel provided ($A_{v(prov)}$) beyond that required by design ($A_{v(req)}$) (Reference 3, Sec. 12.5(d)) and a potential 40% reduction of development strength due to ASR (Reference 4, Table 4) is:

$$l_{d(ASR)} = 1.4 l_d \frac{A_{v(req)}}{A_{v(prov)}}^{15} \tag{14}$$

In a case where the required development length is not provided, the allowable stress on the rebar must be reduced. The relationship between the required development length and the allowable rebar stress $f_{y(eff)}$ is:

$$\frac{l_{d(ASR)}}{f_y} = \frac{l_{d(prov)}}{f_{y(eff)}} \tag{15}$$

Solving the relationship in Eq. 15 for $f_{y(eff)}$ results in:

$$f_{y(eff)} = \frac{l_{d(prov)} f_y}{l_{d(ASR)}} \tag{16}$$

Substituting $f_{y(eff)}$ for f_y in Eq. 12 and considering a 23% strength increase relative to the ACI Code Equations (as justified in Reference 5) results in:

$$v_s = 1.23 \frac{A_{v(prov)} f_{y(eff)}}{b_w s} \tag{17}$$

[14] The variable $\sqrt{f'_c}$ carries the units psi and the coefficient carries the units $\frac{1}{in}$.
[15] See Section 5.2 of this calculation for a discussion of the applicability of the factor $A_{v(req)}/A_{v(prov)}$ to reinforcement which requires development for full f_y.

Substituting the right side of Eq. 16 for $f_{y(eff)}$ results in:

$$v_s = 1.23 \frac{A_{v(prov)}\left(\frac{l_{d(prov)} f_y}{l_{d(ASR)}}\right)}{b_w s} \tag{18}$$

Substituting the right side of Eq. 14 for $l_{d(ASR)}$ results in:

$$v_s = 1.23 \frac{A_{v(prov)}\left(\frac{l_{d(prov)} f_y}{1.4 l_d \frac{A_{v(req)}}{A_{v(prov)}}}\right)}{b_w s} \tag{19}$$

Simplifying the expression:

$$v_s = \left(\frac{1.23}{1.4}\right)\left(\frac{A_{v(prov)}{}^2}{A_{v(req)}}\right)\left(\frac{l_{d(prov)} f_y}{l_d}\right)\left(\frac{1}{b_w s}\right) \tag{20}$$

Substituting the right side of Eq. 13 for l_d and setting A_b equal to $A_{v(prov)}$ results in the reinforcement shear capacity for a wall width equal to the rebar spacing:

$$v_s = \left(\frac{1.23}{1.4}\right)\left(\frac{A_{v(prov)}{}^2}{A_{v(req)}}\right)\left(\frac{l_{d(prov)} f_y}{\frac{0.04 A_{v(prov)} f_y}{\sqrt{f'_c}}}\right)\left(\frac{1}{b_w s}\right) \tag{21}$$

Simplifying the expression:

$$v_s = (0.879)\left(\frac{A_{v(prov)}}{A_{v(req)}}\right)\left(\frac{l_{d(prov)}\sqrt{f'_c}}{0.04}\right)\left(\frac{1}{b_w s}\right) \tag{22}$$

The effect of the potential strength reduction and the documented code conservatism on the steel component of the shear capacity is a factor of 0.879 on capacity, or a 13.8% reduction in margin. The 13.8% margin reduction is less than the arithmetic sum of the 40% reduction and the 23% increase. Therefore, it is conservative to use 17% as the screening criterion.

B

Normalized Hilti Kwik Bolt Capacities

This appendix provides capacities for Hilti Kwik Bolt and Kwik Bolt 2 designs in service at Seabrook Station. The anchor capacities are normalized to the theoretical capacity using the same method applied to test results performed at FSEL as part of this assessment (Reference 9.2.7). Plots of normalized anchor capacity as a function of embedment depth, shown in Figure B-1 and Figure B-2, are discussed in Section 7.5 of this report.

Table B-1 and Table B-2 list the design tensile allowable load for the range of Hilti Kwik Bolt and Kwik Bolt 2 sizes and embedment depths specified for use at Seabrook Station. These design allowable loads were developed by applying a safety factor of four to the mean tested failure load for the anchor. Based on this, the mean anchor capacity is determined by multiplying the design load by SF=4, as shown in Table B-1 and Table B-2. The theoretical anchor capacity is calculated using the following equation, which is explained in Section 7.4.2.

$$N_b = \frac{\psi_c k \sqrt{f'_c} h_{ef}^{1.5}}{F_m}$$

where:

N_b	=	Concrete breakout capacity for a single anchor remote from edges (lb_f)
ψ_c	=	1.0 for cracked concrete
k	=	17 for expansion anchors
f'_c	=	Specified 28-day concrete compressive strength (3,000 psi)
h_{ef}	=	Effective embedment depth (in)
F_m	=	0.7; factor to correct from 5% fractile to mean failure. This ratio represents standard industry practice, and is based on typical sample sizes and coefficients of variation for breakout test.

Table B-1. Hilti Kwik Bolt Normalized Capacities

Diameter (in)	Embedment Depth (in)	Design Allowable Tensile Load (lb)[1]	Design Tensile Capacity (lb)[2]	Theoretical Capacity (lb)[3]	Normalized Capacity[4]
0.5	2.25	1,255	5,020	4,489	1.12
	2.5	1,440	5,760	5,258	1.10
	2.75	1,625	6,500	6,066	1.07
	3.5	2,055	8,220	8,710	0.94
	4.5	2,315	9,260	12,698	0.73
	5.5	2,540	10,160	17,158	0.59
0.625	2.75	1,500	6,000	6,066	0.99
	3.5	1,920	7,680	8,710	0.88
	4.0	2,145	8,580	10,641	0.81
	4.5	2,375	9,500	12,698	0.75
	5.5	2,730	10,920	17,158	0.64
	6.5	3,005	12,020	22,044	0.55
0.75	3.25	2,290	9,160	7,794	1.18
	4.0	2,890	11,560	10,641	1.09
	5.0	3,525	14,100	14,872	0.95
	6.0	3,975	15,900	19,550	0.81
	7.0	4,600	18,400	24,635	0.75
1.00	4.5	3,750	15,000	12,698	1.18
	5.0	4,300	17,200	14,872	1.16
	6.0	5,130	20,520	19,550	1.05
	7.0	5,200	20,800	24,635	0.84
1.25	5.5	5,250	21,000	17,158	1.22
	6.5	6,090	24,360	22,044	1.11
	7.0	6,465	25,860	24,635	1.05
	7.5	6,840	27,360	27,321	1.00
	8.5	7,465	29,860	32,964	0.91
	9.5	8,000	32,000	38,949	0.82

Notes:
1. Reference 9.6.5
2. Design Capacity = Design Allowable Load x 4 (Safety Factor)
3. Theoretical capacity based on mean failure in cracked concrete (See text above).
4. Normalized Capacity = Design Capacity / Theoretical Capacity

Table B-2. Hilti Kwik Bolt 2 Normalized Capacities

Diameter (in)	Embedment Depth (in)	Design Allowable Tensile Load (lb)[1]	Design Tensile Capacity (lb)[2]	Theoretical Capacity (lb)[3]	Normalized Capacity[4]
0.25	1.125	308	1,370	1,587	0.78
	2.0	556	2,690	3,762	0.59
	3.75	625	2,990	9,660	0.26
0.375	1.625	613	3,420	2,755	0.89
	2.5	1,206	5,830	5,258	0.92
	4.25	1,300	6,645	11,655	0.45
0.5	2.25	1,231	5,355	4,489	1.10
	3.5	2,000	8,195	8,710	0.92
	6.0	2,163	8,830	19,550	0.44
0.625	2.75	1,750	7,750	6,066	1.15
	4.0	2,668	11,335	10,641	1.00
	7.0	3,250	13,850	24,635	0.53
0.75	3.25	2,175	9,100	7,794	1.12
	4.75	2,875	15,985	13,771	1.13
	8.0	4,625	21,970	30,099	0.61
1.00	4.5	3,800	15,200	12,698	1.20
	6.0	5,625	22,500	19,550	1.15
	9.0	7,188	28,750	35,915	0.80

Notes:

1. Reference 9.6.5
2. Design Capacity = Design Allowable Load x 4 (Safety Factor)
3. Theoretical capacity based on mean failure in cracked concrete (See text above).
4. Normalized Capacity = Design Capacity / Theoretical Capacity

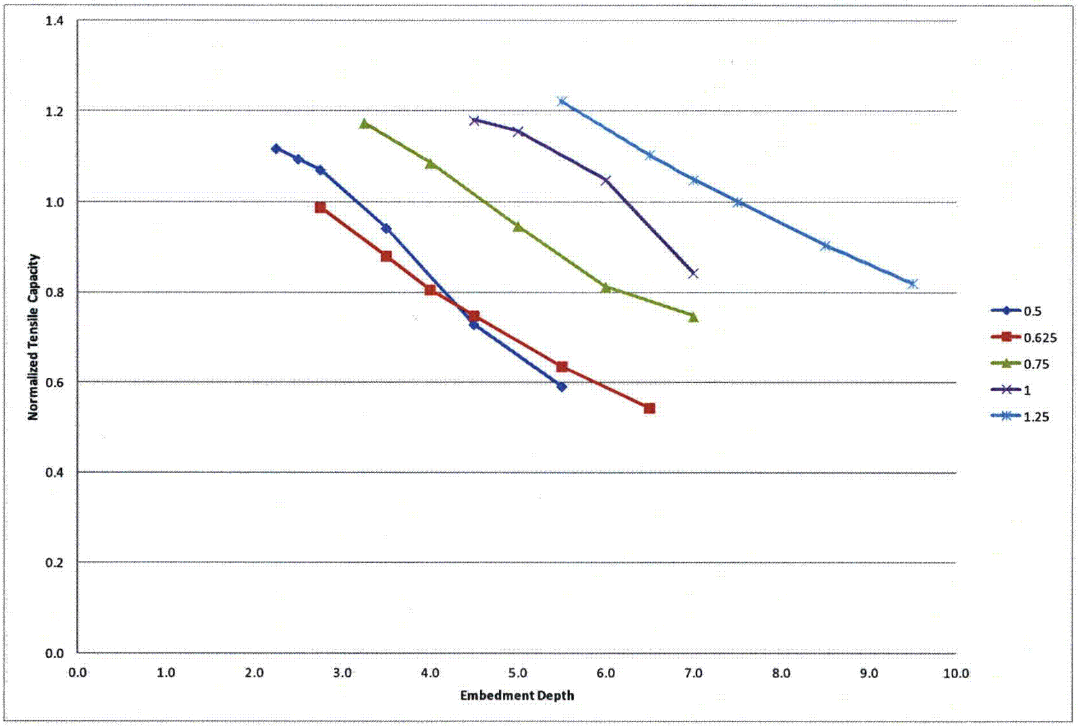

Figure B-1. Hilti Kwik Bolt Normalized Tensile Capacity

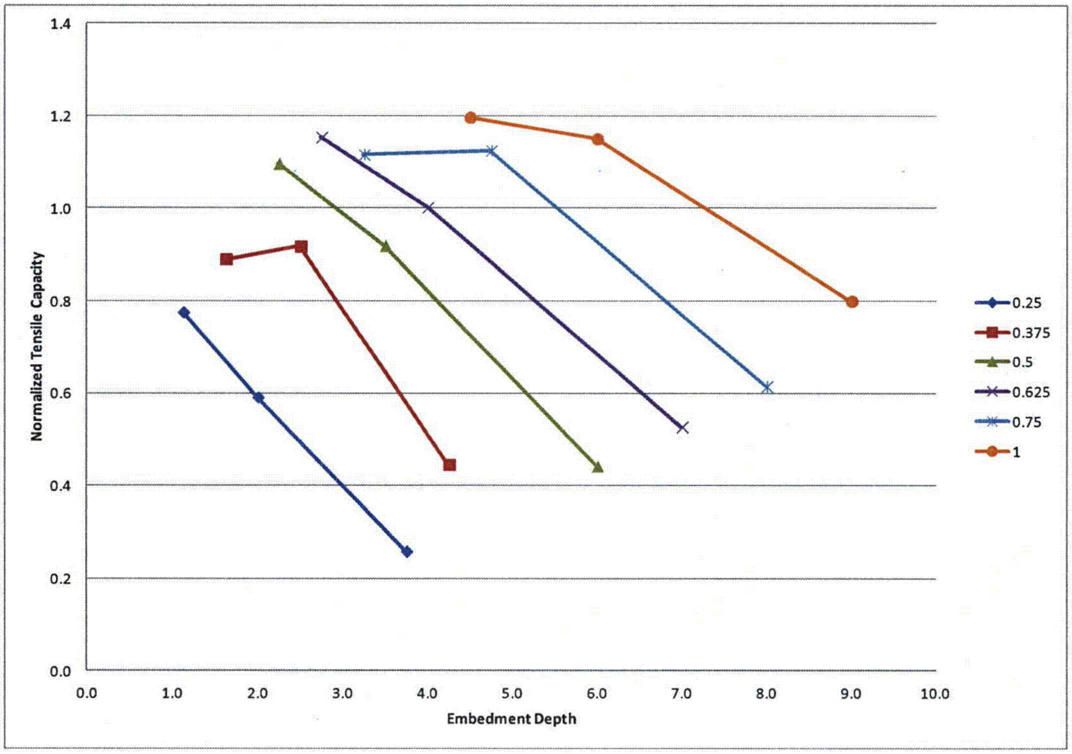

Figure B-2. Hilti Kwik Bolt 2 Normalized Tensile Capacity

FP 100716 Supplement I
Supplement to MPR Report 3727
Seabrook Station: Impact of ASR on Concrete Structures and Attachments

Discussion

FP 100716 (MPR Report -3727) "Seabrook Station: Impact of Alkali-Silica Reaction on Concrete Structures and Attachments" was prepared as an interim assessment to address the impact of ASR at Seabrook. The structural assessment applied a conservative reduction to certain ASR susceptible design parameters. Results of the conservative assessment identified ten locations (one of which has six local areas of concern for a total of fifteen) where there may not be sufficient margin to satisfy the applicable design requirements per ACI 318-71 (FP 100716, Table 6-6). This supplement identifies additional available margin to assure structural integrity.

The additional available margin, based on the existing design basis calculations, at the nine locations identified was computed in Calculation C-S-1-10168. The approach used the design basis calculations as an analysis template and quantified the available margin based on removing load factors applied to the load dead load, live load, hydrodynamic and seismic loadings. Where appropriate the approach used the 28 day compressive strength, based on field cylinder break tests, to compute a higher allowable stress. Either one or both approaches was used in calculating the available margin.

FP 100716 Supplement I
Supplement to MPR Report 3727
Seabrook Station: Impact of ASR on Concrete Structures and Attachments

Table of Margin Assessments

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<tr><th>Wall</th><th>Evaluation</th><th>ASR Effect</th><th>Capacity (FP 100716)</th><th>Demand (C-S-1-10168)</th><th>Margin vs ASR Effect Reduction</th></tr>
<tr><td colspan="6">RHR Vault, Various Rooms</td></tr>
<tr><td>El. (1) 45' 4' Ext Wall</td><td>Out of Plane Shear</td><td>Concrete Shear Capacity Reduce 25%</td><td>v_c = 109 psi</td><td>v_c = 58.3 psi</td><td>46.5% > 25% reduction</td></tr>
<tr><td colspan="6">Emergency Feedwater Pumphouse, Room EFST1</td></tr>
<tr><td>East</td><td>Vertical Reinf. for in-plane Moment El. 0' to 27'</td><td>Embedment & Splice Length Increased 17%</td><td>A = 3.16 in^2/ ft</td><td>A = 1.55 in^2/ ft</td><td>51% > 17% reduction</td></tr>
<tr><td colspan="6">RCA Tunnel</td></tr>
<tr><td>NE Corner Tunnel</td><td>Vertical Reinf. Flexure and Compression</td><td rowspan="3">Embedment & Splice Length Increased 40%</td><td>P_c = 26.5 kip</td><td>P_c = 13.9 kip</td><td>47.6% > 40% reduction</td></tr>
<tr><td>NE Corner Tunnel</td><td>Horizontal Reinf. Shear</td><td>A = 0.88 in^2/ ft</td><td>A = 0.54 in^2/ ft</td><td>39% ~ 40% reduction (within conservative methods of determining the reduction)</td></tr>
<tr><td>West Wall Core Bore RCAW-12</td><td>Flexure and Tension</td><td>P_c = 22.0 kip</td><td>P_c = 10.1 kip</td><td>54% > 40% reduction</td></tr>
</table>

Table of Margin Assessments
(Continued)

Wall	Evaluation	ASR Effect	Capacity (FP 100716)	Demand (C-S-1-10168)	Margin vs ASR Effect Reduction
Diesel Generator Building, Room DG102					
East	Flexure	Embedment & Splice Length Increased 17%	$A = 2.08\ in^2/ft$	$A = 1.41\ in^2/ft$	47.5% > 17% reduction
Cooling Tower					
South, El 32' to 39', Cols A - D	Horiz. reinf. for in-plane shear, bending and tension	Embedment & Splice Length Increased 17%	$A = 1.56\ in^2/ft$	$A = 1.25\ in^2/ft$	20% > 17% reduction
South, El. (-) 8' to 21'	Out of Plane Shear	Concrete Shear Capacity Reduce 25%	$v_c = 111$ psi	$v_c = 73.6$ psi	34% > 25% reduction
South, El. 21' to 45'	Vert. reinf. for bending	Embedment & Splice Length Increased 17%	$A = 0.79\ in^2/ft$	$A = 0.54\ in^2/ft$	32% > 17% reduction
South, El. >50', Cols D-K	Vert. reinf. for bending and tension	Embedment & Splice Length Increased 17%	$A = 0.79\ in^2/ft$	$A = 0.53\ in^2/ft$	33% > 17% reduction