SEABROOK

December 11, 2017

10 CFR 50.90
Docket No. 50-443
SBK-L-17204

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

Seabrook Station
Response to Request for Additional Information Regarding License Amendment Request Related to Alkali-Silica Reaction (CAC No. MF8260; EPID L-2016-LLA-0007)

References:

1. NextEra Energy Seabrook, LLC letter SBK-L-16071, "License Amendment Request 16-03, Revise Current Licensing Basis to Adopt a Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction," August 1, 2016 (ML16216A240).

2. NRC, "Request for Additional Information Regarding License Amendment Request Related to Alkali-Silica Reaction (CAC No. MF8260)," August 4, 2017 (Accession Number ML17214A085).

3. NextEra Energy Seabrook, LLC letter SBK-L-17156, "Response to Request for Additional Information Regarding License Amendment Request 16-03 Related to Alkali-Silica Reaction (CAC No. MF8260)," October 03, 2017.

4. NRC, "Seabrook Station, Unit No. 1 - Request for Additional Information Regarding License Amendment Request Related to Alkali-Silica Reaction (CAC No. MF8260; EPID L-2016-LLA-0007)," October 11, 2017 (Accession Number ML17261B217).

In Reference 1, NextEra Energy Seabrook, LLC (NextEra Energy Seabrook) submitted License Amendment Request 16-03, requesting an amendment to the license for NextEra Energy Seabrook. Specifically, the proposed change revises the NextEra Energy Seabrook Updated Final Safety Analysis Report (UFSAR) to include methods for analyzing Seismic Category I structures with Alkali-Silica Reaction (ASR) affected concrete.

In Reference 2, the NRC requested additional information to complete the review of the NextEra Energy Seabrook License Amendment Request 16-03.

In Reference 3, NextEra Energy Seabrook submitted letter SBK-L-17156 and provided additional information requested in Reference 2.

In Reference 4, the NRC requested additional information to complete the review of the NextEra Energy Seabrook License Amendment Request 16-03, following the submittal in Reference 3.

Enclosure 1 provides NextEra Energy Seabrook's response to the NRC's Request for Additional Information (RAI) concerning the License Amendment Request related to Alkali-Silica Reaction.

Enclosure 2 provides supporting information referenced as Attachment 1 and Attachment 2 in the response to RAI D-8.

Enclosure 3 provides the revised NextEra Energy Seabrook Updated Final Safety Analysis Report (UFSAR) Section 1.8 and 3.8 markups as referenced in the RAI responses within Enclosure 1.

Enclosure 4 provides the methodology document for the analysis of seismic category I structures with concrete affected by Alkali-Silica Reaction for Seabrook Station.

This supplement does not alter the conclusion in Reference 1 that the change does not involve a significant hazards consideration pursuant to 10 CFR 50.92, and there are no significant environmental impacts associated with this change.

No new or revised commitments are included in this letter.

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

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

Executed on December 11, 2017.

Sincerely,

NextEra Energy Seabrook, LLC

Eric McCartney

Eric McCartney
Regional Vice President - Northern Region

Enclosures:

Enclosure 1 – Response to Request for Additional Information Regarding License Amendment Request 16-03 Related to Alkali-Silica Reaction (CAC No. MF8260; EPID L-2016-LLA-0007)

Enclosure 2 – Attachment 1 and Attachment 2, Referenced from Enclosure 1, D-8 Response.

Enclosure 3 – NextEra Energy Seabrook Updated Final Safety Analyses Report Markup of Sections 1.8 and 3.8 – Design of Structures, Components, Equipment and Systems – Design of Category I Structures

Enclosure 4 – Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction for Seabrook Station

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Seabrook Station
Response to Request for Additional Information Regarding License Amendment Request Related to Alkali-Silica Reaction (CAC No. MF8260; EPID L-2016-LLA-0007)

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Enclosure 1 to SBK-L-17204

Response to Request for Additional Information Regarding License Amendment Request Related to Alkali-Silica Reaction (CAC No. MF8260; EPID L-2016-LLA-0007)

RAI-D2

Background

The LAR requests approval of a generic methodology for analyzing and evaluating alkali silica reaction (ASR)-affected structures. LAR Section 3.3.2 states that a "Stage Three: Detailed Evaluation" considers cracked section properties, self-limiting secondary stresses, and the redistribution of structural demands when sufficient ductility is available; however, no detail is provided on the implementation of these methods.

Issue

The implementation of the analysis methods stated to be used in the Stage Three portion of the proposed method of evaluation are not clearly explained, and their implementation may constitute a deviation from the analysis methods in the current licensing basis. The LAR does not request to use analysis methods that deviate from the current licensing basis, nor provide technical justification supporting the use of these methods. Furthermore, there is not sufficient guidance provided in the LAR explaining how the methods will be applied in a consistent, repeatable manner as a generic methodology.

Request

Provide a detailed explanation of how the Stage Three analysis methods will be implemented in a consistent, repeatable manner. If the method of evaluation includes departures (or is modified or supplemented) from the existing design code of record, these deviations should be identified, and a technical justification should be provided of how the proposed alternative provides an acceptable method of complying with applicable NRC regulations or portions thereof. Update the LAR and the UFSAR to incorporate any changes based on this RAI response.

NextEra Energy Seabrook Response to RAI-D2

NextEra Energy Seabrook has prepared a “Methodology Document” (Enclosure 4) that defines in detail the analysis and evaluation procedures for all three stages of analyses described in the LAR 16-03 such that they may be implementable in a consistent and repeatable manner by a knowledgeable structural engineer. The Methodology Document provides details of structural inspections, modeling, analysis, acceptance criteria, threshold monitoring, and criteria for further analysis or structural modification when the threshold monitoring parameters are reaching their limits.

The Methodology Document defines the acceptance criteria for structural evaluation to meet the intent of the original codes of record and identifies deviations from the codes of record with technical justification for these deviations. Implementing the Methodology Document and meeting the acceptance criteria therein demonstrates the structure meets the intent of the original design codes of record and achieves the structural safety reliability indices consistent with the original design. The following text identifies the requested deviations to the codes of record which are considered as supplements to the codes of record:

Supplement 1 - Consideration of ASR loads: The UFSAR load and load combinations tables 3.8-1, 3.8-14 and 3.8-16 were modified in LAR 16-03 to consider the ASR load and load factors for calculating the total demands on structures affected by ASR.

Basis: ASME 1975 [Reference 2] and ACI 318-71 [Reference 1] codes do not address the requirement to consider any special loading for reinforced concrete structures that are impacted by ASR behavior. The concrete ASR growth causes stresses in reinforcement and concrete, and member forces due to deformation compatibility between members expanding due to ASR. SGH 160268-R-01 [Reference 3] defines the ASR load and associated load factors to be combined with the original factored load combinations to provide safety margins comparable to those provided by the original factored loads in the original codes of record.

Supplement 2 – Code acceptance criteria: Strength of reinforced concrete sections affected by ASR can be calculated using the codes of record (ASME 1975 and ACI 318-71) and the minimum specified design concrete strength, provided that through-thickness ASR expansion is within the limits stated in report MPR-4273 [Reference 4].

Basis: Report MPR-4288 [Reference 5] provides the basis for Supplement 2; the report conclusions are supported by the MPR/FSEL large-scale test programs described in report MPR-4273.

Supplement 3 – Shear-friction capacity for members subjected to net compression: The shear-friction capacity for member subjected to net compression can be calculated using procedures defined in Building Code Requirements for Structural Concrete (ACI 318-83 Section 11.7.7).

Basis: The shear friction capacity defined by ACI 318-71 Section 11.15 does not address members subjected to sustained compression. Later versions of ACI 318 and ACI 349 [Reference 6] provide provisions for members subject to permanent net compression when computing shear-friction capacity. The provisions for calculation of shear-friction capacity for members subject to net compression are provided in the following ACI Codes; ACI 318-83 and -02 Section 11.7.7, ACI 318-08, and -11 Section 11.6.7, ACI 318-14 Section 22.9.4.5, and ACI 349-85 and -01 Section 11.7.7. In addition, each of the referenced ACI codes similarly limits the nominal shear stress (or strength), which, in effect, restricts the benefit of permanent net compression. ACI 318-71 Section 11.15.3 limits the maximum nominal shear stress to 0.2f'c or 800 psi; more recent code editions such as ACI 318-83 Section 11.7.5 limit the maximum nominal shear strength to 0.2f'c*Ac or 800*Ac, where Ac is the area of concrete section resisting shear transfer. Both version of ACI 318-71 and -83 also use the same strength reduction factors, Φ, for shear.

Supplement 4 – Flexural Cracked Section Properties: The ratio of cracked over uncracked moment of inertia for flexural behavior can be calculated using ACI 318-71 equation 9-4 or it is acceptable to define the cracked moment of inertia as 50% of the gross moment of inertia as discussed below.

Basis: The flexural stiffness of members with structural cracks or expected to develop structural cracks reduces compared to uncracked section properties. It should be noted that the structural cracks are those due to external loading and not microcracks associated with ASR behavior. The ratio of cracked to uncracked (gross) moment of inertia of 0.5 is consistent with provisions of ACI 318-14 Section 6.6.3.1.2, ASCE 43-05 Table 3-1 [Reference 7], and ASCE 4-16 Table 3-2 [Reference 8]. Additionally, a review of 24 in. deep sections at Seabrook Station structures with 3 in. concrete cover and reinforcement configurations ranging from No. 8 bars at 12 in.

spacing (ratio = 0.0032) to No. 11 bars at 6 in. spacing (ratio = 0.0128) indicates that the ratio of cracked to uncracked gross moment of inertia ranges from 16% to 47%. The first ratio represents minimum allowable flexural reinforcement per ACI 318-71 Section 10.5.1, and the second ratio represents 80% of the maximum allowable flexural reinforcement per ACI 318-71 Section 10.3.2. This review indicates that using a ratio of 50% for reinforced sections in analysis is reasonable or conservative, since flexural demands from ASR tend to decrease as the flexural stiffness decreases. The review also indicates that benefit can be achieved by explicitly calculating and using the cracked section moment of inertia in the analysis. While recent editions of ACI 318 limit the maximum reinforcement ratio via other provisions than contained in ACI 318-71, the effect is similar, and therefore the comparison to the 50% ratio is justified.

Supplement 5 – Axial and Shear Cracked Section Properties: Axial and shear cracking reduces the corresponding stiffness of a structural member. The effect of cracking on reducing the axial and shear stiffness of structural components may be considered in analysis.

Basis: As the net tension on a concrete section reaches or exceeds the concrete tensile stress limit, the tensile stiffness of concrete section is reduced gradually to account for possible aggregate interlocking behavior, which is conservative compared to abruptly reducing the concrete tensile strength to zero. Gradual reduction of concrete tension stiffness results in larger net tension on a section that needs to be resisted by the rebars and hence results in conservative evaluation of rebar strength evaluation. The axially cracked section properties can be calculated based on the stress-strain relationship defined in Methodology Document Appendix A which is based on ACI 224.2R92 [Reference 9] and Lu, Yuan [Reference 10]. The shear cracked section properties can be calculated based on the shear retention factor defined in Methodology Document Appendix A which is based on Červenka V [Reference 11].

References for RAI-D2:

1	ACI 318	Building Code Requirements for Structural Concrete
2	ASME	ASME Boiler & Pressure Vessel Code 1975
3	SGH 160268-R-01	NextEra Energy Seabrook SBK-L-16071, Enclosure 4, SG&H Report 160268-R01, "Development of ASR Load factors for Seismic Category I Structures (including Containment) at Seabrook Station, Seabrook, NH," Revision 0 (Seabrook FP#101039)
4	MPR-4273	NextEra Energy Seabrook SBK-L-16071, Enclosure 6, MPR-4273, Revision 0, "Seabrook Station – Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction," July 2016. (Seabrook FP#101050) (Proprietary)
5	MPR-4288	NextEra SBK-L-16071, Enclosure 5, MPR-4288, Revision 0, "Seabrook Station – Implications of Alkali-Silica Reaction on the Structural Design Evaluations," July 2016. (Seabrook FP#101020) (Proprietary)
6	ACI 349	Codes Requirements for Nuclear Safety Related Structures
7	ASCE 43	ASCE Standard ASCE/SEI 43-05, "Seismic Design Criteria for Structures, System, and Components in Nuclear Facilities."
8	ASCE 4	ASCE Standard ASCE/SEI 4-16, "Seismic Analysis of Safety-Related Nuclear Structures."
9	ACI 224.2R-92	ACI Committee 224, *Cracking of Concrete Members in Direct Tension*, ACI 224.2R-92, Reapproved 1997.
10	Lu, Yuan	Lu, Yuan, and Marios Panagiotou. "Three-dimensional cyclic beam-truss model for nonplanar reinforced concrete walls." Journal of Structural Engineering, 2013, 140(3): 04013071.
11	Červenka V.	Červenka V., Jendele L., and Červenka J., ATENA Program Documentation, Part 1, Theory. Dec. 2016, pp 29.

RAI-D3

Background

LAR Section 1.0 proposes to revise the UFSAR to include methods for analyzing seismic Category 1 structures with concrete affected by ASR. LAR Section 1.0 states that the Seabrook seismic Category I structures, other than containment, were designed in accordance with ACI 318-71, while the containment was designed in accordance with ASME Section III, Division 2, 1975 Edition. LAR Section 3.3.2 states that for the "Stage Three: Detailed Evaluation":

> The structure is evaluated using strength acceptance criteria in ACI 318-71 for reinforced concrete consistent with UFSAR Section 3.8.4.5. In the Stage Three evaluation, consideration is given to cracked section properties, self-limiting stresses, and the redistribution of structural demands when sufficient ductility is available.

ACI 318-71, Section 8.6, includes provisions for moment redistribution of negative moments calculated by elastic theory at the supports of continuous flexural members. This code section specifies a moment redistribution limit as a function of the tension reinforcement ratio and reinforcement ratio producing balanced conditions, subject to an upper limit of 20 percent. ACI 318-71 allows the use of such moment redistribution only when the section at which the moment is reduced is so designed that the tension reinforcement ratio is equal to or less than 0.5 times the reinforcement ratio producing balanced conditions, as defined in Section 10.3.3 of the code (i.e., the section design has sufficient ductility). The NRC staff notes that no deviations or alternatives from ACI 318-71 provisions (along with sufficient justification) have been proposed in the LAR.

Issue

From the NRC staff review of the Containment Enclosure Building (CEB) Evaluation Report, it is not clear how the moment redistribution approach described in the report meets the criteria in ACI 318-71 or other accepted concrete codes. Specifically, the staff notes the following:

a. The LAR indicates that the design is performed in accordance with ACI 318-71 and considers the redistribution of structural demands when sufficient ductility is available. The CEB report indicates that moment redistribution is used when the axial-flexure (PM) interaction demands exceed their code capacity; however, the CEB report does not appear to address ACI 318-71, Section 8.6, or other requirements to be met for using moment redistribution.

b. The capacity of concrete structures to absorb inelastic rotations at plastic-hinge locations is not unlimited; therefore, the analysis should consider not only the amount of rotation required at critical sections to achieve the assumed degree of moment distribution, but also the rotation capacity of the members at those sections to ensure it is adequate. It does not appear there are specific acceptance criteria for the structural adequacy of a concrete section that develops a plastic hinge. In the case of the CEB, only the strain in the reinforcing steel was calculated.

c. It is not clear if there is a limit on redistribution with the current moment redistribution approach or how the process works if subsequent iterations cause excess moments to occur in the first set of location(s) (e.g., what occurs if convergence to a valid set of results everywhere is not achieved).

Request

a. Explain with sufficient technical detail how the proposed moment redistribution approach meets specific requirements of ACI 318-71 that may be applicable. Provide technical justification for any portions that deviate from the code requirements. Provide the technical basis for concluding that ACI 318-71 covers the use of moment redistribution for structures receiving a Stage Three analysis. Identify any industry codes, standards, guides, published research, and test data that substantiate the deviations.

b. Provide the acceptance criteria and technical basis for the criteria for the structural adequacy of a concrete section that develops a plastic hinge. As an example, acceptance criteria for design parameters to demonstrate the structural adequacy may include limitations on the steel to concrete ratio, permissible ductility ratio (in terms of total displacements of the concrete section) or rotational capacity, and ensuring that flexure, not shear, controls the design.

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

d. Update the LAR, UFSAR section markups, and other Seabrook design documents, as applicable, consistent with the responses to this RAI.

NextEra Energy Seabrook Response to RAI-D3, Request "a", "b", "c", and "d"

NextEra Energy Seabrook hereby amends the analysis method described in LAR 16-03 to restrict moment redistribution to be in accordance with the provisions of ACI 318-71 Section 8.6. NextEra Energy Seabrook is revising the CEB evaluation to consider cracked section properties instead of the moment redistribution method used in Revision 0 of this calculation.

For reference, the moment redistribution used in Revision 0 of the Containment Enclosure Building (CEB) evaluation results in conservative results, since the demands were calculated based on simulation of the field observed deformation using uncracked properties. For locations where the conservatively calculated axial force (P) and moment (M) exceeded the P-M strength, the excess moment demand above strength was redistributed to confirm that it would not adversely impact the P-M evaluation at other locations within the CEB structure. For further conservatism, the moment redistributions were taken from a concentrated small section cut area.

RAI-D4

Background

During a June 5, 2017, to June 9, 2017, site audit, the NRC staff reviewed CEB evaluation report, SG&H 150252-CA-02, Revision 0, Seabrook FP#100985, July 2016. Appendix L of this report describes the procedure to implement moment redistribution in the finite element model. It describes the "simplified moment redistribution" method, where after applying all the factored load(s) for the load combination, the excess moment above the code section capacity is determined. Then, the excess moment is redistributed in a separate analysis. Superposition of the two analyses is used to determine the result after initial moment redistribution. If there are locations where the moment exceeds the code section capacity, the process is repeated until all locations fall under the code section capacity.

Issue

Based on the NRC staff's review of the procedure, it would appear to be necessary that all analyses in the sequence be performed using the same structural model and boundary conditions, since results from different analyses are superposed.

Request

To ensure that the NRC staff has correctly interpreted the procedure described in Appendix L, confirm that the same structural model and boundary conditions are used for all analyses in the sequence. If this is not the case, describe the different models used and provide the technical basis for using different models, including the validity of superposing results obtained from different models.

NextEra Energy Seabrook Response to RAI-D4

As described in the response to RAI-D3, NextEra Energy Seabrook hereby amends the analysis method described in LAR 16-03 to restrict moment redistribution to be in accordance with the provisions of ACI 318-71 Section 8.6. NextEra Energy Seabrook is revising the CEB evaluation to consider cracked section properties instead of the moment redistribution method used in Revision 0 of this calculation (Seabrook FP#100985). Appendix L of the calculation will be superseded in the revised calculation.

RAI-D5

Background

In LAR Section 3.3.2, the licensee states that original design loads will be combined with the self-straining loads from ASR expansion, and a three-stage process is proposed for analyzing ASR-affected structures. In this discussion, a "threshold limit" is introduced for monitoring ASR effects for each structure. The threshold limit is the value for each monitoring element at which the factored self-straining load equals the design limit when combined with the factored design-basis loads. In a Stage One analysis, an acceptance limit of 90 percent is placed upon the threshold limit. In a Stage Two analysis, a limit of 95 percent is used. In a Stage Three analysis, a limit of 100 percent is used.

Issue

For Stage One and Stage Two analyses, existing design-basis analysis methods are used, and the threshold limit represents the margin remaining between the code allowable limits and the design-basis loading, plus the self-straining loads from ASR.

In Stage Three, additional analysis methods are employed (100-40-40, cracked section properties, moment redistribution), and a threshold factor is applied to account for future ASR expansion. Section 7.3 of the CEB evaluation report states, "The threshold factor is selected to be the largest factor in which the structure meets evaluation criteria using the approaches described in this calculation," and a threshold factor of 1.2 is reported for the CEB. However, as discussed in Section 7.6.2 of the CEB evaluation report, Stage Three analysis uses an iterative process that allows moments to be redistributed to demonstrate that demands meet code capacities.

Since the demands upon the structure are being modified in Stage Three analyses, it is not clear what exactly the threshold factor represents or how it will be selected in future Stage Three analyses.

Request

a. Clarify what the threshold factor represents in Stage Three analyses and how the factor will be determined for future analyses (i.e., is the factor always set at 1.2 or does it depend on each analysis).

b. Explain if there is a limit imposed on the extent of analysis that can be used to modify the demands upon a structure and if this impacts the specification of the threshold factor. Provide a technical justification for the adequacy of the limit or justification for the lack of a limit.

NextEra Energy Seabrook Response to RAI-D5, Request "a"

The threshold factor is design margin expressed as the amount which ASR loads can increase beyond values used in the calculations such that the structure or structural component will still meet the allowable limits of the code of record as supplemented (as discussed in RAI-D2 response). To qualify a structure, the code allowable limits must be larger than the factored design-basis loading plus factored current ASR loading. This margin is being used for demands associated with future ASR growth, so called here as threshold limit. It is an outcome of the evaluation, not an input to the analysis methodology approach. The threshold factor is calculated by increasing the factored ASR loads (factored ASR load multiplied by threshold factor) until the combination with the factored original design load becomes equal to or less than the code allowable limits. A unique threshold factor is calculated for each building based on the available margin, and is used to establish threshold limits for structural monitoring parameters. The threshold factor may be revised based on further analysis, by using additional inspection and measurement data, and/or using a more refined structural analysis method without reducing the code inherent margin of safety.

NextEra Energy Seabrook Response to RAI-D5, Request "b"

There is no limit on re-evaluation provided the evaluation satisfies the code of record with supplements defined in responses RAI-D2. Moment redistribution is limited per the ACI code of record. Re-evaluations would use additional inspection and measurement data, and/or use a more refined structural analysis method in accordance with the Methodology Document. Structural modification may be used to reestablish margin of safety.

RAI-D6

Background

Standard Review Plan 3.7.2 references Regulatory Guide (RG) 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," for acceptable methods for combining the effects of three directions of earthquake loading. For response spectrum analysis only, RG 1.92, Revision 3, Regulatory Position 2.1, states that either the square-root-of-the-sum of-the-squares (SRSS) or 100-40-40 methods are acceptable.

Part B, "Discussion" (page 7), of RG 1.92, Revision 3, states:

> The 100-40-40 percent rule was originally proposed as a simple way to estimate the maximum expected response of a structure subject to three-directional seismic loading for response spectrum analysis, and is the only alternative method for spatial combination that has received any significant attention in the nuclear power industry.

In the LAR, the licensee has proposed a change to the licensing basis (UFSAR markup) permitting use of the 100-40-40 combination method in accordance with RG 1.92, Revision 3, in addition to the SRSS combination method for combining the effects of three directions of earthquake loading. The licensee's proposed UFSAR markup specifically states:

> A procedure for combining the three spatial components of an earthquake for seismic response analysis of nuclear power plant structures, systems, and components (SSCs) that are important to safety is presented in Subsection C.2.1. The Response Spectrum Method that uses the 100-40-40 percent combination rule, as described in Regulatory Position C.2.1 of this guide, is acceptable as an alternative to the SRSS method. (emphasis added)

Issue

Based on review of the CEB evaluation report and discussions with the licensee during the June 5, 2017, to June 9, 2017, site visit, it is unclear to the NRC staff that the licensee is applying the 100-40-40 spatial combination method in accordance with RG 1.92, Revision 3, and the Seabrook UFSAR markup, which identify that the 100-40-40 spatial combination method is applicable to response spectrum analysis. The CEB calculation instead uses an equivalent static analysis with the 100-40-40 method.

Request

a. Clarify whether the 100-40-40 method will be implemented in equivalent static analyses for ASR-affected structures. If so, provide the technical basis for using the method in conjunction with equivalent static analysis.

b. Clarify the UFSAR markup and the LAR to describe the specific conditions under which the 100-40-40 spatial combination method may be implemented.

NextEra Energy Seabrook Response to RAI-D6, Request "a"

NextEra Energy Seabrook hereby amends the analysis method described in the LAR to eliminate the use of 100-40-40 method as an option for combining the effects of seismic loading in three directions. Accordingly, NextEra Energy Seabrook is revising the CEB evaluation to use the Square Root of Sum of the Squares (SRSS) method in conjunction with the equivalent static analysis method to be consistent with the original design calculation procedures.

For reference, the seismic accelerations for Seabrook in the original design calculations were calculated using the response spectra analysis method. The original design calculation used the static equivalent method in conjunction with the SRSS method to compute structural responses. RG 1.92 Section C2.1 [Reference 1] allows the use of 100-40-40 for calculating responses when response spectra analysis are used. Use of static equivalent method that was used in the original design process, and allowed by ASCE 4-98 [Reference 2] and ASCE 4-16 [Reference 3] is not specified or restricted to be used in conjunction with either of SRSS or 100-40-40 methods. However, NextEra Energy Seabrook will, for all building calculations, use the seismic procedures of SRSS and static equivalent method that were used in the original design calculations.

NextEra Energy Seabrook Response to RAI-D6, Request "b"

As discussed above, NextEra Energy Seabrook hereby amends the analysis method described in LAR 16-03 to eliminate the use of 100-40-40 method as an option for calculating seismic demands for any Seismic Category I structures at Seabrook. The UFSAR markup Section 1.8 and Section 3.7(B).2.1 have been revised to use the original SRSS methods and provided in Enclosure 3.

References for RAI-D6:

1	RG 1.92	USNRC Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Revision 2, July 2006
2	ASCE 4-98	ASCE Standard ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures."
3	ASCE 4-16	ASCE Standard ASCE/SEI 4-16, "Seismic Analysis of Safety-Related Nuclear Structures."

RAI-D7

Background

Standard Review Plan 3.7.2 references RG 1.92 for acceptable methods for combining the effects of three directions of earthquake loading. Part B, "Discussion" (page 7), of RG 1.92, Revision 3, states:

> The results of the 100-40-40 spatial combination have been compared with the SRSS spatial combination. Generally, they indicate that the 100-40-40 combination method produces higher estimates of maximum response than the SRSS combination method by as much as 16 percent, while the maximum under-prediction is 1 percent.

The UFSAR markup makes a similar statement regarding the conservatism of the 100-40-40 method; however, the LAR supplement dated September 30, 2016 (response to Item 4 in Enclosure 1), indicates that the switch from SRSS to 100-40-40 is intended to gain additional margin to accommodate the effects of ASR.

Issue

It is not clear how the 100-40-40 method is being implemented, since the UFSAR states it is generally conservative, while the LAR supplement, as above, indicates that the use of 100-40-40 is intended to gain margin. Consequently, the staff requested and reviewed, via the online audit portal, sample 100-40-40 calculations prior to the June 5-9, 2017 site visit. This subject was also discussed during the site visit. Based on its review and the audit discussions, the staff has identified the following issues with the reviewed sample calculation:

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

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

b. The calculation includes two loads, E_o and H_e. Based on the method of implementing 100-40-40, the combined $E_o + H_e$ in some cases is less than E_o alone. Inherent in a calculation that produces lower responses for the combination of E_o and H_e, compared to E_o alone, is the potential assumption that there is a defined phase relationship between the two loads. This assumption does not appear to be justified in the calculation.

Request

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

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

<u>NextEra Energy Seabrook Response to RAI-D7, Request "a"</u>

NextEra Energy Seabrook hereby amends the analysis method described in LAR 16-03 to eliminate the use of 100-40-40 method as an option for combining the effects of seismic loading in three directions. Accordingly, NextEra Energy Seabrook is revising the CEB evaluation to no longer use the 100-40-40 method and instead use the Square Roots of Sum of Squares (SRSS) method with the static equivalent analysis procedure to be consistent with original design calculations performed. For conditions with multiple components (e.g., axial force and moment P-M interaction), components are being calculated by the SRSS method. The SRSS calculated ±P and ±M will be used for P-M interaction evaluation. The UFSAR markup Section 1.8 and Section 3.7(B).2.1 have been revised to use the original SRSS methods [Enclosure 3].

For reference, the 100-40-40 methodology used for calculating seismic responses in Revision 0 of Containment Enclosure Building (CEB) evaluation is consistent with methodology defined in ASCE 4-98 [Reference 2] and ASCE 4-16 [Reference 3]. The 100-40-40 methodology used for CEB results in consistent values compared to RG 1.92 [Reference 1] for each individual component of response such as axial force, P, moment, M, or shear, V. RG 1.92 only discusses one component of the response and does not provide any guidance on responses with multiple values like P-M interaction. The three directional components of earthquake motion are uncorrelated and therefore the maximum responses due to orthogonal directions are not expected to occur at same time. The use of 100-40-40 for evaluation with multiple components of responses such as P-M interaction is more realistic than using the maximum of both responses due to orthogonal directions. For the PM responses at a horizontal section of the CEB, it is expected that the maximum P occurs due to vertical excitation, while maximum moment about horizontal axis, M, occurs due to horizontal excitation. Therefore 100-40-40 P-M interaction that was used for the CEB better reflect the structural responses than conservatively using maximum of P and maximum of M from SRSS. However, NextEra Energy Seabrook will, for all building calculations, use the SRSS method when computing multiple components of responses.

<u>NextEra Energy Seabrook Response to RAI-D7, Request "b"</u>

For the CEB calculation it is considered that the seismic inertia force, E_o, and soil pushing the embedded part of the CEB, H_e, are in-phase. This consideration results in maximum base shear and overturning moment since the static equivalent method and the SRSS responses are used. The out-of-plane bending response in the CEB is influenced by the presence of large penetrations, and the location of applied loads including dynamic soil loads. The dynamic soil response and inertial response may counteract each other at limited localized locations. However, since the analyses are repeated for all three input seismic motion including the opposite directions, any of these localized locations will be covered since the results are enveloped.

References for RAI-D7:

1	RG 1.92	USNRC Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Revision 2, July 2006
2	ASCE 4-98	ASCE Standard ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures."
3	ASCE 4-16	ASCE Standard ASCE/SEI 4-16, "Seismic Analysis of Safety-Related Nuclear Structures."

RAI-D8

Background

Seabrook UFSAR Sections 3.8.4.3 and 3.8.4.5 provide definitions and structural acceptance criteria, respectively, of normal operating (service) load conditions and unusual load conditions for Seismic Category 1 structures (other than containment). The Seabrook UFSAR Subsection 3.8.4.3.b.1, "Normal Load Conditions," states:

> Normal load conditions are those encountered during testing and normal operation and are referred to in the standard review plan as service load conditions. They included dead load, live load and anticipated transients, and loads occurring during normal startup and shutdown, and .... Normal loading also includes the effect of an operating basis earthquake and normal wind load. Under each of these loading combinations the structures were designed so that stresses are within elastic limits.

The corresponding structural acceptance criteria for normal load conditions in UFSAR Subsection 3.8.4.5.a states: "Structures were proportioned to remain within the elastic limits under all normal loading conditions described in Subsection 3.8.4.3. Reinforced concrete structures were designed in accordance with ACI 318 strength method, which insures flexural ductility by limiting reinforcing steel percentages and stresses. Similar current licensing basis information is provided in UFSAR Subsections 3.8.3.3 and 3.8.3.5, and 3.8.1.3 and 3.8.1.5, for containment internal structures and containment, respectively.

The UFSAR markup for Sections 3.8.1.3(f), 3.8.3.3(e), and 3.8.4.3a.1(e), incorporated ASR load as a design-basis self-straining load, and states, in part: "ASR loads are passive and therefore occur during normal operation, shutdown conditions, and concurrently with all extreme environmental loads." Thus, ASR is a service load that exists on a day-to-day basis during normal operating or service conditions of the plant.

As required by GDC 1, where generally recognized codes and standards are used, they shall be evaluated to determine their applicability, adequacy, and sufficiency and shall be supplemented or modified as necessary to assure a quality product in keeping with the required safety function. It is noted that ACI 318-71, the construction code-of-record for seismic Category 1 structures (other than containment) at the Seabrook Station, did not consider ASR effects in its code provisions and that ASR is not a typical design-basis load. The design philosophy in ACI 318-71 includes considerations of strength, as well as serviceability (e.g., Sections 9.1.2, 9.5) requirements intended to limit conditions that may adversely affect the strength or serviceability of the structure at service load levels.

Issue

LAR Section 3.2.2, under the title "Reinforcement Steel Strain," states:

> The expansion of concrete from ASR-induced cracking imposes a tensile strain on steel reinforcement within the affected material. For structures designed to ACI 318-71, the design code allows for reinforcement strains beyond the yield point of the steel bars for flexural elements to prevent brittle compression failure of the concrete in bending. The added strain to the reinforcement should be evaluated in conjunction with the strains imposed by other loads on the structure.

As noted above, the design code allows for reinforcement strains beyond yield for determining the flexural capacity in strength design for comparison against ultimate (factored) loads. However, under realistic (unfactored) normal operating or service load conditions, the design code ensures stresses and strains will remain within elastic limits through serviceability considerations. ASR expansion is a self-straining service load whose progression has potential for straining the reinforcement beyond yield under normal operating conditions. The progression or sustenance of the prestressing effect with ASR expansion and concrete cracking is not well understood or documented, especially if rebar is strained beyond yield due to ASR.

As required by the structural design in the Seabrook UFSAR, stresses and strains in the structures shall be maintained within elastic limits under normal operating load conditions. Potential yielding of the rebar due to ASR under service conditions could be indicative of a marked change in the behavioral response of a structure, could impact structural capacity, and can render assumptions of linear-elastic behavior in the structural analyses (including seismic analyses) unjustified. However, the proposed method of structural evaluation for ASR-affected structures, which includes provisions for cracked sections and redistribution of structural demand, does not appear to include a verification of the concrete and rebar stresses and strains based on realistic behavior under normal operating conditions (including ASR) that would ensure they remain within elastic limits, as required by the UFSAR.

Request

Explain, with sufficient technical detail, how the proposed method of evaluation (Stage One, Stage Two, and Stage Three) for ASR-affected structures verifies that the stresses and strains in the concrete and reinforcement remain within elastic limits based on realistic behavior under normal operating (service) load conditions, including ASR load. Update the UFSAR markup and the LAR as necessary based on the response.

NextEra Energy Seabrook Response to RAI-D8

Seismic Category I structures that were designed to ACI 318-71 (i.e., Seismic Category 1 structures other than Containment) [Reference 1] that are analyzed using approaches described in the Methodology Document and meet the acceptance criteria therein will respond elastically under realistic (unfactored) normal operating or service load conditions. This conclusion is based on the following considerations.

- Structures that meet the ACI 318-71 strength method requirements with the supplements listed in RAI-D2 respond elastically under realistic (unfactored) normal operating or service load conditions.
- Parametric studies demonstrate that rebar stress responses remain in elastic range until the member is subjected to significant loading in addition to high ASR loading.
- Evaluation of calculation results for a sample of eight Seabrook Station Category I structures (that are representative of all Category I structures) confirms that, under realistic normal operating or service loading conditions, the rebar stresses remain below yield limit and compressional strain in the concrete remain significantly below ACI 318-71 Code maximum usable strain for compression.

Since the structures meeting the analysis and acceptance criteria described above ensures that the response remains elastic under normal operating or service load conditions, NextEra Energy Seabrook concludes that the check described in the last bullet above does not need to be incorporated into the Methodology Document and does not need to be performed for the remaining structures at Seabrook Station.

The following subsections provide detailed discussions regarding the conclusion and the bulleted items above.

ACI 318-71 Strength Design Method:

Seismic Category I structures other than Containment were originally designed to meet the requirements of ACI 318-71 using the strength design method. This is in accordance with the UFSAR and the requirements of NUREG 0800 SRP Section 3.8.4 [Reference 2] .

In the strength design method a member is qualified when the demand forces due to factored loads are less than or equal to the section strength (nominal strength multiplied by strength reduction factors Φ). There is no specific requirement to compute and check rebar stresses or concrete strains under factored load or realistic (unfactored) normal operating or service load conditions, since the code compliance will inherently keep the stresses and strains below acceptable limits.

ACI 318-71 Section 9.5 requires floors and roofs to have adequate stiffness to control deflections under service loads (unfactored loads). Report MPR-4273 [Reference 3], which summarizes the MPR/FSEL large-scale test programs, concluded that the overall stiffness of ASR-affected structures does not reduce compared to the original design conditions. Therefore, the deflection limits that were met by the original design will still be met regardless of whether the section is now affected by ASR up to the limit provided in MPR report MPR-4288 [Reference 4].

Although there is no specific requirement in the strength design method to check stresses and strains to be less than elastic limit under service loads, this is achieved by using the load factors and resistance factors in the strength design method. As an example, a structural member designed with Grade 60 steel reinforcement to resist axial tension caused by dead load using a load factor of 1.4 and resistance factor of $\phi = 0.9$ would have the maximum steel stress of (60 ksi / 1.4)*0.9 = 39 ksi under service loading. Therefore, if a member is not affected by ASR and self-straining effects, and if the design meets the acceptance criteria of the ACI 318-71 strength design method, no further stress checks are needed; and both stress and strain in concrete and steel are expected to be less than the elastic limit under service loading conditions.

Parametric Studies:

Members with internal ASR expansion have additional tensile stress in steel rebar and compressive stress in concrete due to self-straining ASR effects. These stresses develop because the steel rebar is restraining the concrete from expanding. Two parametric studies were performed to examine the effects on rebar stress of increasing ASR expansion in combination with external loads. Although these parametric studies do not represent all section configuration for structures at Seabrook Station, they provide insights on the response of structural members subjected to the combined effect of internal ASR and external original design loadings that are relevant to the behavior of the structures at Seabrook Station.

Parametric Study 1

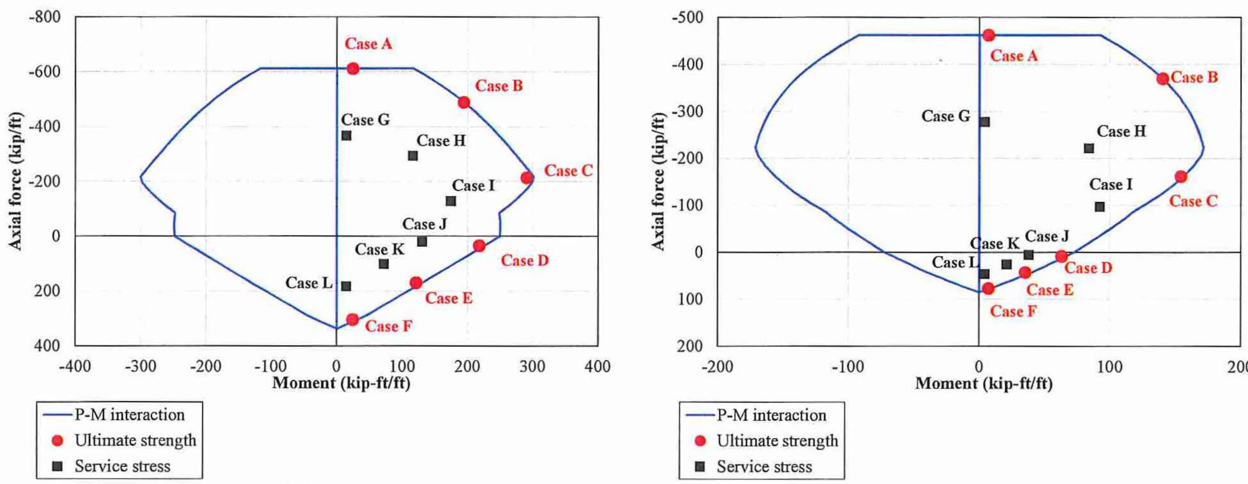
Parametric Study 1 evaluates the effects of increasing ASR expansion on rebar stress for a member already loaded with external loadings. The study is performed for a typical 2 ft thick wall/slab with two different levels of reinforcement ratios as follows:

- A member with a relatively high reinforcement ratio (1.3%, equivalent to $0.6\rho_b$):
  - #11@6in. (e.g. walls of Main Steam & Feed Water Pipe Chase – East).
  - Design capacity: ϕMn = 242 kip-ft/ft, ϕPn = 337 kip/ft

- A member with a relatively low reinforcement ratio (0.3%, equal to ρ_{min}):
  - #8@12in. (e.g. wall of Control Room Makeup Air Intake).
  - Design capacity: ϕMn = 70.1 kip-ft/ft, ϕPn = 85.3 kip/ft

The member is first subjected to the combined axial force (P) and bending moment (M) due to external loads. Then internal ASR strains are increased from 0.0 to 2.0 mm/m. It should be noted that the 2.0 mm/m ASR strain used here is just an upper limit set for this parametric study and is not a limit set by MPR/FSEL large scale test programs. The ASR load simulates the self-straining behavior of placing the steel in tension and concrete in compression. Several cases of P-M combinations are considered as shown in P-M plots in Figure 1 and summarized in Table 1:

- Cases A through F represent factored load level forces near the section strength limits (demands on the P-M curve), and
- Cases G through L represent the expected service load conditions (loads at about 60% of Cases A through F).



(a) (b)

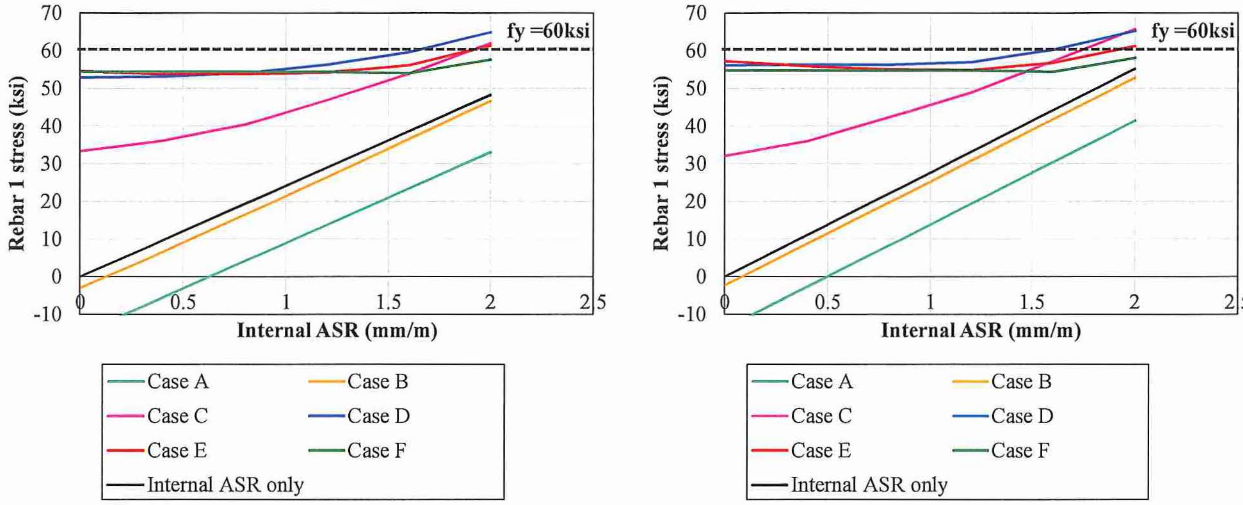
Figure 1: P-M Combinations Cases (a) Section design with high steel ratio, (b) Section design with low steel ratio

Table 1: P-M Combinations Cases

Case	Load Level	Axial Force (P)	Moment (M)
A	Factored load level forces near the section strength	High Compression	Low
B		Medium Compression	Medium
C		Low Compression	High
D		Low Tension	High
E		Medium Tension	Medium
F		High Tension	Low
G	Service Load	High Compression	Low
H		Medium Compression	Medium
I		Low Compression	High
J		Low Tension	High
K		Medium Tension	Medium
L		High Tension	Low

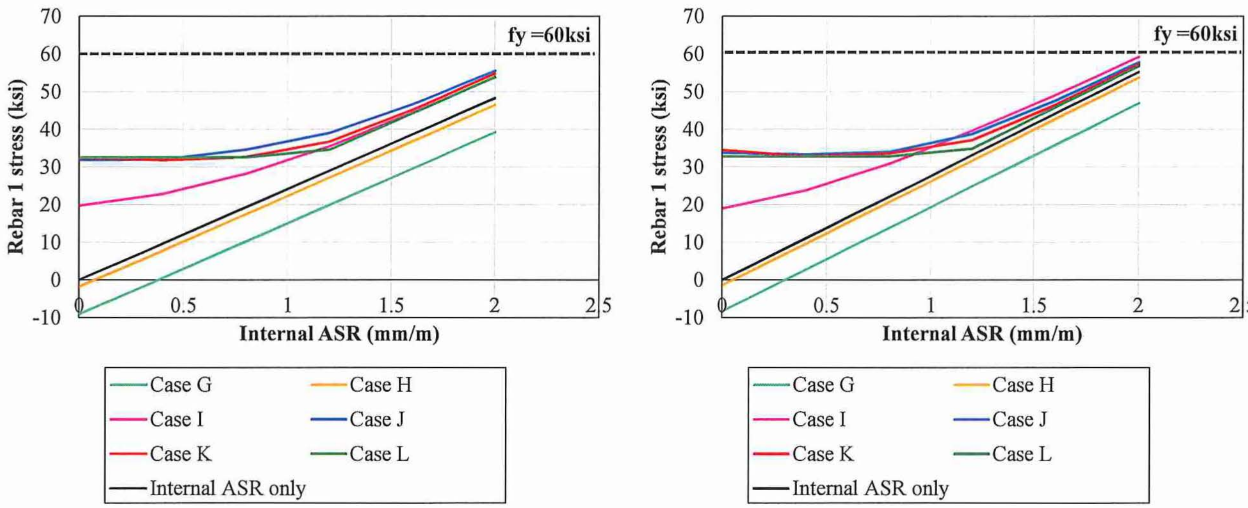
The relationship between the rebar stresses with the increasing level of ASR strain when the member has already been subjected to factored external load levels (Cases A through F) is shown in Figure 2 for members with high and low reinforcement ratios. The stress in the rebar is computed using a fiber section method, in which the section is divided into twenty thin layers with concrete represented with compressive stiffness only (no tension stiffness) and the reinforcement is represented with linear elastic behavior. The internal ASR expansion is modeled by applying autogenous (initial) strain to concrete fibers that will be restrained by the rebars. Details of the procedures and calculations leading to the reinforcement stress plots are presented for representative Case I in the Attachment 1.

The relationship between the rebar stresses and the increasing level of ASR strain when the member has already been subjected to service external load levels (Cases G through L) is shown in Figure 3 for members with high and low reinforcement ratios.



(a) (b)

Figure 2: Stress in tension rebar of typical 2 ft thick member subjected to factored load level P-M and increasing ASR strain (a) section design with high reinforcement ratio, (b) section design with low reinforcement ratio



(a) (b)

Figure 3: Stress in tension rebar of typical 2 ft thick member subjected to service load level P-M and increasing ASR strain (a) section design with high reinforcement ratio, (b) section design with low reinforcement ratio

The following can be concluded from the results shown in Figures 2 and 3:

- The tensile stress in the rebar only increases significantly after ASR expansion becomes sufficient to close concrete tensile cracks around the tensile rebars. The reason for this is because concrete cracks must close before ASR growth can increase the rebar strain or stress.
- The level of ASR which causes tensile stress in steel rebar to increase is a function of the level of external load. For sections with already high levels of rebar tensile stress (factored level loads), high levels of ASR are needed to cause additional tensile stress in steel rebar.
- Stresses and strains in steel rebar are less than the elastic limits at service load

conditions, provided that ASR strain is less than 2 mm/m.

- The effect of ASR on rebar stresses is similar for both high and low reinforcement ratios.

Parametric Study 2

Parametric Study 2 evaluates the effects of increasing external bending moment on rebar stress of a section that already experienced self-straining stresses due to different levels of ASR strains. The study is performed for the same two design sections of Parametric Study 1 with high and low reinforcement ratios. The analyses are performed using the same fiber section method used for Parametric Study 1.

The member is first subject to the ASR expansion; then external bending moments (without external axial load) are increased. Several levels of ASR strain, ε_{Sa}, are considered:

- Case 1: ε_{Sa} = 0.0 mm/m - None
- Case 2: ε_{Sa} = 0.5 mm/m - Severity Zone 1 upper limit
- Case 3: ε_{Sa} = 1.0 mm/m - Severity Zone 2 upper limit
- Case 4: ε_{Sa} = 1.5 mm/m - Severity Zone 3
- Case 5: ε_{Sa} = 2.0 mm/m - Severity Zone 4 lower limit

The relationship between the rebar stresses and the level of external bending moment is shown in Figure 4 for members with relatively high and low reinforcement ratios.

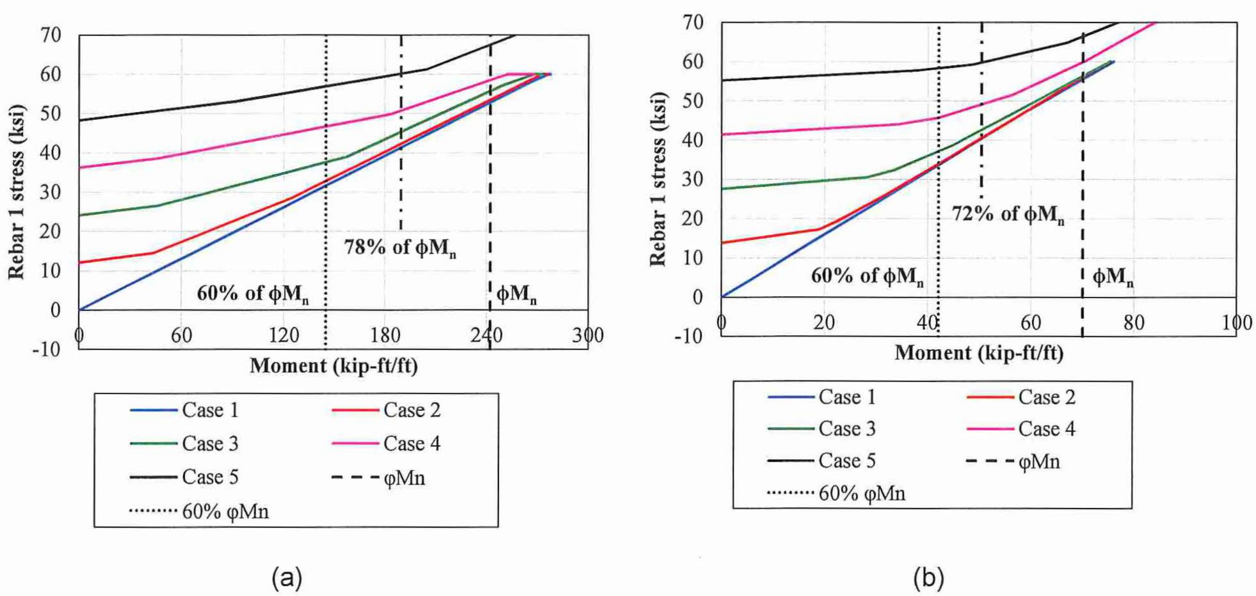


Figure 4: Stress in tension rebar of typical 2 ft thick member subjected to ASR and increasing moment (a) section design with high reinforcement ratio, (b) section design with low reinforcement ratio

The following can be concluded from the results shown in Figure 4:

- Self-straining concrete compressive stress due to ASR prestressing effect must unload before rebar tensile stress significantly increases. The reason for this is because the concrete section that is still in compression due to prestressing is much stiffer than the steel section with the reinforcement ratio less than the reinforcement ratio at balanced conditions.
- The rebars do not yield even at the design strength if the internal ASR expansion is below

1.5 mm/m (Case 4). This is because of the use of the resistance factor when computing the design strength.

- The rebars do not yield even if the internal ASR expansion reaches to 2.0 mm/m if the moment due to external load is less than 70% of design strength (0.7ϕMn).

Check of Seabrook Structural Responses:

To confirm that the above conclusions apply to ASR-affected Seabrook seismic category I structures for which evaluations have been completed using the strength provisions of ACI 318, rebar tensile stresses and concrete compressive stresses are calculated for realistic (unfactored) normal operating (service load) conditions. Results are provided for the following structures:

- Control Room Makeup Air Intake structure (CRMAI)
- Residual Heat Removal Equipment Vault structure (RHR)
- Containment Enclosure Building (CEB)
- Enclosure for Condensate Storage Tank (CSTE)
- Main steam and feed water west pipe chase and Personnel Hatch (WPC/PH)
- Containment Equipment Hatch Missile Shield structure (CEHMS)
- Containment Enclosure Ventilation Area (CEVA)
- Safety-Related Electrical Duct Banks and Manholes (EMH) W01, W02, W09, and W13 through W16

The above listed structures well represent the Seismic Category I structures at Seabrook Station subjected to ASR loadings, and structures evaluated using the three different analysis stages defined in LAR 16-03. The above structures include:

- Three structures (CRMAI, RHR, and CEB) evaluated using Stage Three analysis.
- Two structures (CSTE and WPC/PH) evaluated using Stage Two analysis.
- Three structures (CEHMS, CEVA, and EMH) evaluated using Stage One analysis.

These structures are subjected to different levels of internal in-plane ASR expansions:

- CRMAI, RHR, and CEHMS structures with maximum in-plane expansion higher than 0.7 mm/m.
- CEB and CSTE structures with in-plane expansion between 0.35 to 0.7 mm/m.
- WPC/PH, EMH, and CEVA with maximum in-plane expansion less than 0.35 mm/m.

The CEB, CRMAI, RHR, and CSTE represent structures with significant variation between ASR expansions between structural members (such as wall to wall, wall to slab), and within regions (such as regions of shells, or regions on slabs). The CEB and RHR represent structures with deformations and structural cracking and are subjected to expansion effects of concrete backfill, while CRMAI, CSTE, and WPC/PH represent structures with no or minimal concrete backfill expansion behavior. The WPC/PH and RHR represent structures subjected to ASR expansions that are impacted by structural connectivity to other structures. The RHR, CRMAI, and EMH represent fully embedded structures, while CEB, CEVA, and CSTE represent partially embedded structures. The CEB with cylindrical shape and major openings, and CSTE with cylindrical shape with connecting walls and slabs represent structures with complex geometry, while CRMAI represents a structure with simple geometry.

The stresses for the above-listed structures that are qualified by meeting the strength method of ACI 318-71 with the supplements listed in RAI-D2 are presented here for realistic (unfactored) normal operating service load conditions and results shown the rebars stresses remain below yield stress and concrete strains remain significantly below code allowable strain limit. It should be noted that the stresses for the north wall of CEVA that do not meet the ACI 318-71 strength method at this time is excluded, since this wall need to be retrofitted. The conceptual upgrades for the north wall of CEVA are being considered to make this wall to conform to ACI 318-71.

The following two load combinations are used to calculate the stresses for normal operating service load conditions:

- $D + L + E + T_o + S_a$ (LC1 – In-situ condition)
- $D + L + E + T_o + E_o + H_e + F_{THR} \cdot S_a$ (LC2 – In-situ condition + OBE + future ASR)

where D is dead load, L is live load, E is lateral earth pressure, T_o is operating temperature, E_o is the operating basis earthquake (OBE), H_e is dynamic earth pressure due to OBE, S_a is ASR load, and F_{THR} is the threshold factor. Operating temperature T_o is only applicable to the WPC/PH. LC1 represents the best estimate of the in-situ condition, and LC2 represents the in-situ condition, plus the OBE and effects of future additional ASR up to the threshold limit.

The stresses are calculated using a fiber section method subjected to internal ASR strain and external loads on the section. The ASR strain is the same ASR expansion used for the structural evaluation and the external loads on this section are based on demands calculated from the structural analysis. Tables 2 and 3 report the maximum steel rebar tensile stress, and concrete compressive stress and strain for service loadings LC1 and LC2, respectively. Attachment 2 provides the detailed calculations for the stress and strains reported in Tables 2 and 3. The stresses provided here represent the maximum stresses for each structure corresponding to members in the design load path which have internal ASR strains. These provide maximum stress and strain for each building, but since the maximum rebar tension and concrete compression for CEB occurs at different location then results for both locations are provided. Although the stresses for members not subject to internal ASR strains are not requested in this RAI, the stresses for these members if they were governing the ASR deformation design evaluation are provided in the Attachment 2. These Tables also list the stage of analyses performed for each structure and the level of internal ASR strain associated with the member for which the stress is reported for.

The maximum tensile stress for LC1 is 46.9 ksi, which occurs in the reinforcement in the East exterior wall of the RHR near the connection to the Primary Auxiliary Building (PAB). The majority of the stresses that develop at this location are due to the RHR connection to PAB. The PAB foundation locally stiffens the connection between the RHR and the PAB which attracts the moment demand about the vertical axis in the east exterior wall of the RHR. In addition, the PAB base slab is subject to uplift pressure from backfill expansion which in turns induces forces in the RHR external walls near the connection. The stresses in the RHR evaluation and as reported here are conservative due to only including a limited model of PAB as connected to RHR which introduces extra overturning moment as well as the expected vertical shear force at this connection. The 46.9 ksi is based on averaging the demand on a section cut of five elements with length of less than 4.5 wall thicknesses of 2 ft. Considering more detail model of the PAB and considering longer section cut at this location would reduce the conservatism in the stress reported here. The maximum tensile stress for rebars in other buildings evaluated here is less than 33 ksi. The maximum compressive strain in concrete is 0.00072 which is significantly less than 0.003 the ACI 318-71 Code maximum usable strain for compression.

The maximum tensile stress for LC2 is 56.5 ksi, which occurs in the reinforcement in the East exterior wall of the RHR at the connection to the Primary Auxiliary Building (PAB). As discussed above the 56.5 ksi stress is conservative since the stress is calculated based on section cut in the

2 ft thick wall only, and because of the finite element modeling truncation of the PAB structure, as discussed above. Note that moment redistribution allowed per ACI 318-71 Section 8.6 has not been used for calculating the reported stress for the RHR exterior wall rebar. The maximum tensile stress for rebars in other buildings evaluated here is less than 45 ksi. The maximum compressive strain in concrete is 0.00091 which is significantly less than 0.003 the ACI 318-71 Code maximum usable strain for compression.

In conclusion, the rebar stresses are below elastic limits for all structures listed here when considering the two realistic unfactored service level loadings. The concrete compressive stress remains below the crushing limit, and the concrete strains are less than 0.001 much less than the 0.003 the ACI 318-71 Code maximum usable strain for concrete compression. The structures presented here as described above well represent the Seismic Category I structures at Seabrook Station that are subjected to ASR expansions. Therefore, the other seismic category I structures do not require explicit evaluation for unfactored service level loads.

Table 2: Stress in rebars of critical components of Seabrook structures (LC1)

Structure	Analysis Stage	ASR (mm/m)	Component	Maximum tensile stress in rebar (ksi)	Maximum compressive stress in concrete (ksi)	Maximum compressive strain in concrete (m/m)
CRMAI	**3**	0.99	Base Mat	27.8	-0.28	-0.00009
RHR	**3**	0.75	East exterior wall	46.9	-1.9	-0.00061
CEB	**3**	0.60	Wall near foundation	27.1	-2.21	-0.00066
		0.10	Wall above Elec. Penet.	24.6	-0.71	-0.00019
CSTE	**2**	0.43	Tank wall	15.8	-0.68	-0.00019
WPC/PH	**2**	0.24	North wall	7.8	-0.07	-0.00002
CEHMS	**1**	0.72	East wing wall	23.4	-0.78	-0.00025
CEVA	**1**	0.31	Base slab	32.8	-0.89	-0.00028
EMH	**1**	0.25	W13/W15 walls	11.2	-0.28	-0.00009

Table 3: Stress in rebars of critical components of Seabrook structures (LC2)

Structure	Analysis Stage	ASR (mm/m) times threshold	Component	Maximum tensile stress in rebar (ksi)	Maximum compressive stress in concrete (ksi)	Maximum compressive strain in concrete (m/m)
CRMAI	**3**	0.99 x 1.4	Base Mat	39.1	-0.33	-0.00011
RHR	**3**	0.75 x 1.2	East exterior wall	56.5	-2.1	-0.00067
CEB	**3**	0.60 x 1.3	Wall near foundation	42.5	-2.68	-0.00085
		0.10 x 1.3	Wall above Elec. Penet.	55.6	-1.33	-0.00037
CSTE	**2**	0.43 x 1.6	Tank wall	26.7	-1.11	-0.00031
WPC/PH	**2**	0.24 x 1.8	North wall	44.4	-1.36	-0.00044
CEHMS	**1**	0.72 x 1.5	East wing wall	41.6	-1.52	-0.00049
CEVA	**1**	0.31 x 3.0	Base slab	44.0	-1.08	-0.00035
EMH	**1**	0.25 x 3.7	W13/W15 walls	27.0	-0.3	-0.000097

References for RAI-D8:

1	ACI 318-71	Building Code Requirements for Structural Concrete, 1971
2	SRP Section 3.8.4	USNR NUREG 0800 Standard Review Plan, Section 3.8.4, "Other Seismic Category I Structures", Rev 1, July 1981
3	MPR-4273	NextEra SBK-L-16071, Enclosure 6, MPR-4273, Revision 0, "Seabrook Station – Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction," July 2016. (Seabrook FP#101050) (Proprietary)
4	MPR-4288	NextEra SBK-L-16071, Enclosure 5, MPR-4288, Revision 0, "Seabrook Station – Implications of Alkali-Silica Reaction on the Structural Design Evaluations," July 2016. (Seabrook FP#101020) (Proprietary)

Enclosure 2 to SBK-L-17204

Attachment 1 and Attachment 2, Referenced from Enclosure 1, D-8 Response.

RESPONSE TO RAI-D8-ATTACHMENT 1

EXAMPLE CALCULATION OF REBAR STRESS FOR A SECTION SUBJECTED TO COMBINED EFFECT OF EXTERNAL AXIAL AND MOMENT AND INTERNAL ASR

1. REVISION HISTORY

Revision 0: Initial document.

2. OBJECTIVE OF CALCULATION

The objective of this calculation is to provide an example calculation of rebar stress used in parametric studies 1 and 2 in response to RAI-D8.

3. RESULTS AND CONCLUSIONS

Table 1 summarizes the tensile stress in rebars corresponding to constant axial force and moment with an increasing ASR expansion. The results are also plotted in Figure 1b. This data is used to draw diagrams similar to what presented in Figure 3b of parametric study 1.

4. DESIGN DATA / CRITERIA

Diagrams presented in the response to RAI-D8 are extracted for two extreme sections one with minimum reinforcement ratio and the other with maximum reinforcement ratio. There is no other criteria.

5. ASSUMPTIONS

5.1 Justified assumptions

The concrete material is represented by compression only elastoplastic material with compressive strain cutoff of 0.003. This simple constitutive model satisfactorily captures the response of concrete in compression because stresses are not near reaching the compressive strength. Attachment 2 Appendix H provides a comparison study between the stresses in rebars of the critical component of two structures (with high and low compressive stress in concrete) computed using two different constitutive models for concrete, namely:

- Accurate model that uses Kent and Park concrete response in compression
- Simple model/idealized model which is an elastoplastic model with compressive stress cutoff at compressive strain of 0.003

The concrete strength in tension is conservatively neglected.

5.2 Unverified assumptions

There are no unverified assumptions.

6. METHODOLOGY

As an example calculation, Case I for a section with high reinforcement ratio is considered. The section is 2ft thick with 3000psi concrete that is reinforced with #11@6in. on both faces. The point corresponding to case I is highlighted on P-M interaction diagram provided in Figure 1. The amount of axial force and moment for Case I are -128.5kip/ft and 174.2kip-ft/ft, respectively.

To calculate the diagram in parametric study 1, the axial force and moment are kept constant while the internal ASR load is increased. Such a diagram is presented in Figure 3 of the response to RAI-D8. For the second parametric study, specific ASR expansion is selected and the amount of moment is increased. The calculation presented here provides an example for both parametric studies. In fact, the loading sequence does not matter.

The stress in rebars is calculated considering the following steps:

1) The geometry including thickness, rebar size, spacing, etc. are provided.
2) The compatibility and equilibrium equations are satisfied for concrete and steel when the concrete undergoes expansion due to internal ASR. Consequently, the initial stresses in concrete and steel are calculated.
3) Appropriate material model are assigned for concrete and steel. Specifically, elastic material for steel and an elastoplastic material for concrete are used.
4) Section is discretized into 20 layers, and appropriate functions are developed to facilitate the calculation of strain and stress at middle of each layer. Steel layers are also used at the center of rebars at each faces.
5) By knowing the value of axial force "P" (P = -128.5kip/ft), the curvature value "φ" is iterated to minimize the difference between the target moment (M = 174.2kip-ft/ft) and the moment from sectional analysis based on inputted axial force and trial curvature.
6) Using the developed functions, the strain and consequently stress are calculated for each steel fiber and at the farthest edge of the concrete compressive fiber.

7. REFERENCES

There are no references

8. COMPUTATION

8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index $\varepsilon_{CI} := 0.8 \frac{mm}{m}$

Threshold factor $F_{thr} := 1$

Material properties

Compressive strength of concrete $f_c := -3ksi$

Young's modulus of concrete $E_c := 3120ksi$

Yield strength of steel $f_y := 60ksi$

Young's modulus of steel $E_s := 29000ksi$

Geometry

Width of fibers $b := 12in$

Total thickness or height $h := 24in$

Area of concrete $A_c := b \cdot h = 288 \cdot in^2$

Area of tensile reinforcement (#8@12 in.) $A_s := 2 \cdot 1.56in^2$

Number of reinforcement in row, e.g. equal to 2 for tensile and compressive $Steel_{Num} := 2$

Depth to reinforcement $d := 20.3in$

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

Initial Guess

Initial mechanical strain in concrete $\varepsilon_{o.conc} := 0$

Initial strain in steel $\varepsilon_{o.steel} := 0$

Given

Compatibility equation $F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$

Equilibrium equation $\left(E_c \cdot A_c\right) \cdot \varepsilon_{o.conc} + \left(E_s \cdot A_s \cdot Steel_{Num}\right) \cdot \varepsilon_{o.steel} = 0$

$ans := Find\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}\right)$

Initial strain in concrete and steel

$\varepsilon_{o.conc} := ans_1 = -1.341 \times 10^{-4}$

$\varepsilon_{o.steel} := ans_2 = 6.659 \times 10^{-4}$

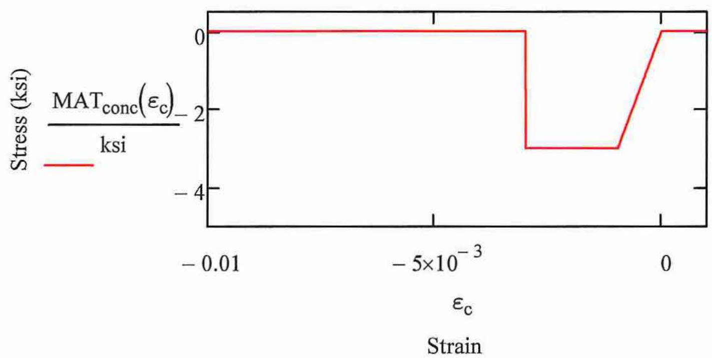
8.2. Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

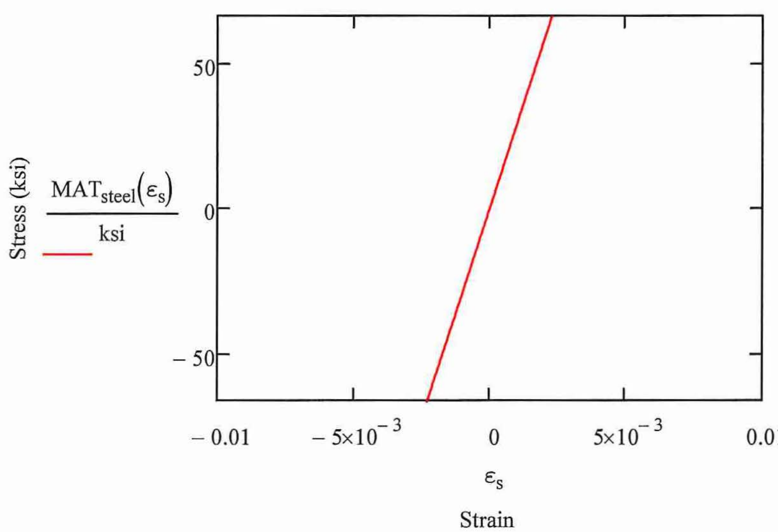
$$MAT_{conc}(\varepsilon) := \begin{cases} 0 & \text{if } \varepsilon > 0 \\ f_c & \text{if } -0.003 \le \varepsilon < \dfrac{f_c}{E_c} \\ 0 & \text{if } \varepsilon < -0.003 \\ \left(E_c \cdot \varepsilon\right) & \text{otherwise} \end{cases}$$



Steel Material Model

Constitutive model for steel

$MAT_{steel}(\varepsilon) := E_s \cdot \varepsilon$



Concrete Fibers

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \frac{h}{Conc_{Num}} = 1.2 \cdot in$

Concrete fiber coordinates
$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\frac{h}{2} + \frac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain
$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress
$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force
$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber
$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -8.3 \cdot in$$
$$Steel_{y_2} := d - \frac{h}{2} = 8.3 \cdot in$$

Area of reinforcement fiber
$$Steel_{As_1} := A_s = 3.12 \cdot in^2$$
$$Steel_{As_2} := A_s = 3.12 \cdot in^2$$

Steel fiber strain
$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress
$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force
$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$$\text{Concrete}_{\sigma} := \text{Conc}_{\sigma}\left(\varepsilon_{o.conc}, 0, 0\right)$$

$$\text{Concrete}_{\sigma_1} = -0.418 \cdot \text{ksi}$$

Initial stress in steel

$$\text{Rebar}_{\sigma} := \text{Steel}_{\sigma}\left(\varepsilon_{o.steel}, 0, 0\right)$$

$$\text{Rebar}_{\sigma_1} = 19.311 \cdot \text{ksi}$$

Axial Equilibrium

$$\text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Conc}_{\text{Num}} \\ \quad \text{ans1} \leftarrow \text{ans1} + \text{Conc}_F\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_i \\ \text{ans2} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Steel}_{\text{Num}} \\ \quad \text{ans2} \leftarrow \text{ans2} + \text{Steel}_F\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_i \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Moment Equilibrium

$$\text{Moment}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Conc}_{\text{Num}} \\ \quad \text{ans1} \leftarrow \text{ans1} + -1 \cdot \text{Conc}_F\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_i \cdot \text{Conc}_{y_i} \\ \text{ans2} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Steel}_{\text{Num}} \\ \quad \text{ans2} \leftarrow \text{ans2} + -1 \cdot \text{Steel}_F\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_i \cdot \text{Steel}_{y_i} \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Solution

Known parameters

Axial force $P := -128.52\text{kip}$

Iteration

Curvature $\phi := 0.000046 \cdot \frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium

$$f(x) := \text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi) - P$$

$$\varepsilon_{cent} := \text{root}(f(x_o), x_o) = -7.471 \times 10^{-5}$$

Sectional forces

$$\text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = -128.52 \cdot \text{kip}$$

$$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \blacksquare \cdot \text{kip} \cdot \text{ft}$$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_\varepsilon := \text{Steel}_\varepsilon(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 9.73 \times 10^{-4} \\ 2.094 \times 10^{-4} \end{pmatrix}$$

$$\text{Rebar}_\sigma := \text{Steel}_\sigma(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 28.217 \\ 6.072 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_F(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 88.036 \\ 18.946 \end{pmatrix} \cdot \text{kip}$$

$$\text{Concrete}_y := \text{Conc}_y$$

Concrete fiber stress and strain

$$\text{Concrete}_\varepsilon := \text{Conc}_\varepsilon(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

$$\text{Concrete}_\sigma := \text{Conc}_\sigma(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \ldots = -7.608 \times 10^{-4}$$
$$+ \text{Concrete}_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete $\sigma_{max.comp} := \text{MAT}_{conc}(\varepsilon_{max.comp}) = -2.374 \cdot \text{ksi}$

9. TABLES

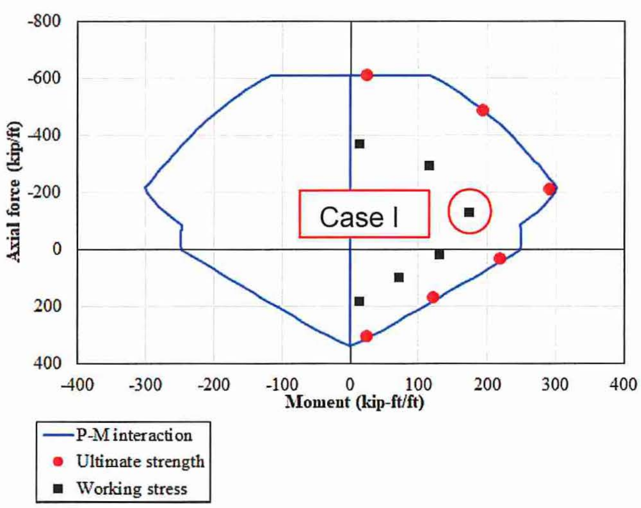
Table 1: Stress in rebars of 2ft thick section with high reinforcement ratio for P=-128.52kip/ft and M=174.24kip-ft/ft

CI (mm/m)	initial stress in concrete (ksi)	Curvature, φ (1/in)*	Total stress in steel (ksi) Rebar 1	Total stress in steel (ksi) Rebar 1	Maximum compressive stress in concrete (ksi)
0	0	0.00007	19.737	-13.961	-2.31
0.4	9.655	0.000056	22.869	-4.089	-2.334
0.8	19.311	0.000046	28.217	6.072	-2.374
1.2	28.966	0.00004	35.511	16.255	-2.457
1.6	38.622	0.000038	44.377	26.084	-2.624
2	48.277	0.0000375	53.851	35.799	-2.821

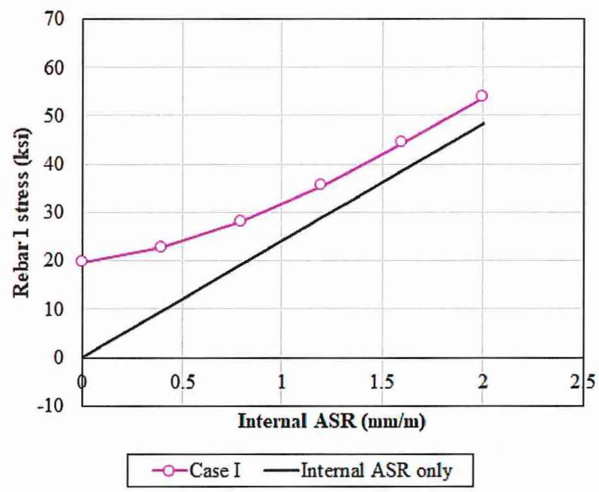
***The curvature needs to be found iteratively to satisfy the moment equilibrium**

Example in Section 8

10. FIGURES



(b) Location of Case I in P-M interaction



(b) Stress in the critical rebar of Case I

Figure 1: Results for Case I

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

		PROJECT NO:	170444
		DATE:	Dec 2017
CLIENT:	NextEra Energy Seabrook	BY:	MR. M. Gargari
SUBJECT:	Evaluation of maximum stress in rebars of Seabrook structures	VERIFIER:	A.T. Sarawit

RESPONSE TO RAI-D8-ATTACHMENT 2

EVALUATION OF MAXIMUM STRESS IN REBARS OF SEABROOK STRUCTURES

1. REVISION HISTORY

Revision 0: Initial document.

2. OBJECTIVE OF CALCULATION AND SCOPE

The objective of this calculation is to evaluate the stress in rebars of the structures at NextEra Energy (NEE) Seabrook Station in Seabrook, New Hampshire for in-situ load combinations considering unfactored normal operating loads when adding the loads due to ASR. All demands are from the ASR susceptibility evaluation of each structure.

The scope of this calculation includes the following structures:

- Control Room Makeup Air Intake structure (CRMAI)
- Residual Heat Removal Equipment Vault structure (RHR)
- Containment Enclosure Building (CEB)
- Enclosure for Condensate Storage Tank (CSTE)
- Main steam and feed water west pipe chase and Personnel Hatch (WPC/PH)
- Containment Equipment Hatch Missile Shield structure (CEHMS)
- Containment Enclosure Ventilation Area (CEVA)
- Safety-Related Electrical Duct Banks and Manholes (EMH) W01, W02, W09, and W13 through W16

3. RESULTS AND CONCLUSIONS

Stress evaluation results are listed below:

- The structure is evaluated for the load combinations listed in Section 4. The load combination listed below controls the calculation of maximum stress in rebars.
  - $D + L + E + T_o + E_o + H_e + F_{THR}.S_a$ (LC2)
- The stress in rebars of all structural components remain below yield strength. The following components give the highest stress in rebars:
  - Rebars along the horizontal strip of east exterior wall of the RHR structure at approximate elevation of -30 ft are stressed to 56.5 ksi subjected to LC2. The high stress is expected to occur in localized area, and therefore, the moment can distributed to mid span in susceptibility evaluation of the structure [3]. In addition, the stresses are expected to less because of the conservatism including a limited model of PAB as connected to RHR as explained in Section 6.3.
  - The maximum axial stress of 55.6 ksi is expected in rebars of the wall above east corner of Electrical Penetration at EL +45 ft subjected to LC2 in CEB.
  - Rebars along the horizontal strip at east wall of CRMAI structure are expected to experience tensile stress as high as 43.3ksi. The CI/CCI value over the walls of the structure is zero, and the induced demands are mainly due to relative expansion of the base mat with respect to walls.
  - Rebars in the east-west direction at the base slab of CEVA are expected to be stressed to 44 ksi if the CI value increases 200% beyond the current state. As explained in Section 6.6, the actual value is expected to be less because of the conservatism in computing unfactored demands due to original loads.

4. DESIGN DATA / CRITERIA

In response to RAI-D8 request, the maximum stress in the rebars of Seabrook structures is calculated and compared with yielding strength of rebars (f_y = 60ksi). In this evaluation, the following in-situ load combinations (also called service load and unfactored normal operating load) are considered:

- $D + L + E + T_o + S_a$ (In-situ condition, LC1)
- $D + L + E + T_o + E_o + H_e + F_{THR}.S_a$ (In-situ condition plus seismic load, LC2)

where D is dead load, L is live load, E is lateral earth pressure, T_o is operating temperature, E_o is the operating basis earthquake (OBE), H_e is dynamic earth pressure due to OBE, and S_a is ASR load. Operating temperature T_o is only applicable to the WPC/PH. For the second in-situ load combination, ASR loads are further amplified by a threshold factor (F_{THR}) to account for the future ASR expansion.

5. METHODOLOGY

To calculate the stress in rebars of structural components subjected to in-situ load combinations, sectional analysis based on fiber section method is used. In this method, the cross section is discretized into fibers (or layers), and an appropriate material model is assigned to each fiber. Figure 1 demonstrates a typical fiber section discretization. The total moment and axial force are calculated by integrating force over all fibers.

The concrete material is represented by compression only elastoplastic material with compressive strain cutoff of 0.003. This simple constitutive model satisfactorily captures the response of concrete in compression because stresses are not near reaching the compressive strength. Appendix H provides a comparison study between the stresses in rebars of the critical component of two structures (with high and low compressive stress in concrete) computed using two different constitutive models for concrete, namely:

- Accurate model that uses Kent and Park concrete response in compression
- Simple model/idealized model which is an elastoplastic model with compressive stress cutoff at compressive strain of 0.003

Both models are schematically depicted in Figure 2a. The concrete strength in tension is conservatively neglected. Reinforcing steel bars are modeled using elastic perfectly plastic material in compression and tension. Figure 2b demonstrates the steel material model used for the section analysis. The initial slope (Young modules) are 29,000 ksi for steel and $57{,}000\sqrt{f'_c}$ for concrete.

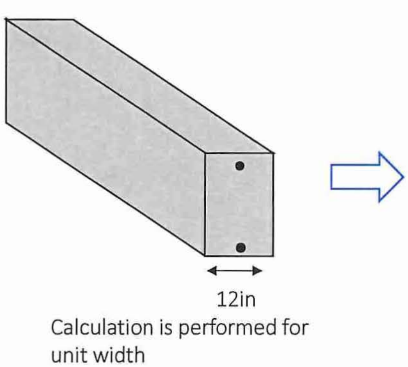
In this evaluation the ASR load effect causes:

- The axial force and bending moment that are induced by ASR expansion of other components (adjacent structural component)

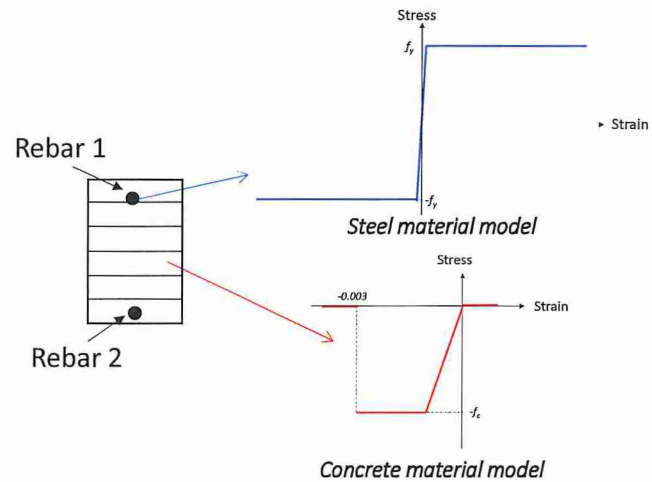
- The internal stress in rebars due to ASR expansion of the component itself

The latter induces tensile stress in rebars and compressive stress in concrete that is called initial stress state. The effect of internal ASR expansion is considered by adding autogenous strain to the concrete and steel material. The input strain magnitude is set to be the ASR strain value measured over the specific component, and the output strains (initial strain in concrete and steel after application of ASR strain) are calculated by satisfying equilibrium and compatibility equations. If a member does not show any sign of internal ASR or the internal ASR expansion of the member was conservatively set equal to zero during ASR susceptibility evaluation of the structure, the initial stress in concrete and rebar are set to zero.

The critical sections that governed the calculation of threshold factor of each structure are selected for the evaluation, and demands due to combined effects of internal ASR expansion and induced ASR expansion of other components are computed with methods used in susceptibility evaluation of the structures. Appendix J provides Run ID logs. These demands are added to the demands subjected to original design loads, and the stress in rebars are calculated.

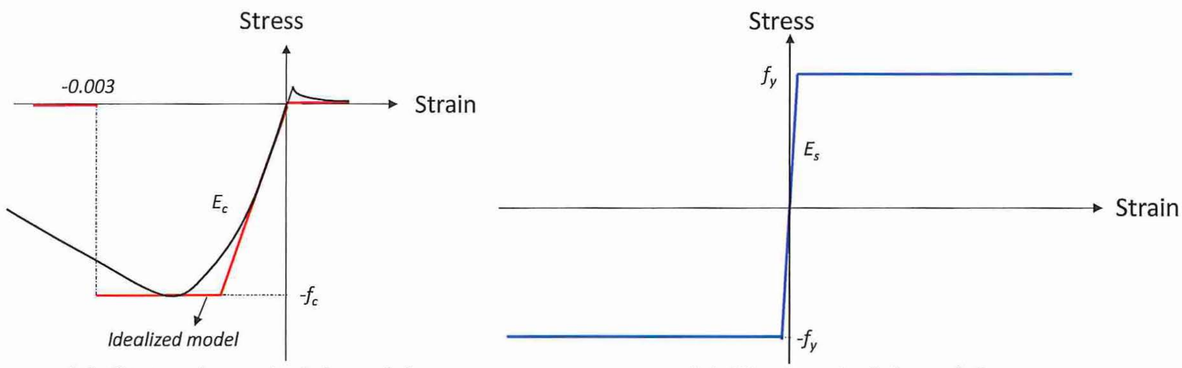


Actual concrete member



Fiber section discretization and material models

Figure 1 – Schematic representation of fiber section method



(a) Concrete material model (b) Steel material model

Figure 2 – Concrete and steel material model

6. ANALYSIS AND EVALUATION COMPUTATIONS

This section summarizes the maximum stress that are computed in rebars and concrete of several Seabrook structures at critical sections.

6.1 Control Room Makeup Air Intake structure

The stress in rebars of the critical components of CRMAI structure that governed the calculation of threshold factor is calculated and presented in Appendix A. Calculation of the threshold factor for the CRMAI structure is primarily governed by axial-flexure interaction along the horizontal strip of the east wall that occurs at the middle of the wall [1]. A threshold factor of 1.4 was determined from evaluation of the CRMAI structure, which indicates that ASR-related demands are amplified by 40% beyond the factored values.

Tables 1 and 2 summarize the stress in rebars of east wall and base mat of CRMAI structure. As can be seen from the table, the maximum axial stress is 43.3 ksi expected to form in a horizontal rebar of the walls close to the interior of the structure. The maximum stress in base mat that has highest ASR expansion within the structure is 39.1 ksi. Both stresses are below the yield strength of rebars.

6.2 Containment Enclosure Building

The stress in rebars at the critical section of the CEB structures is calculated and presented in Appendix B. The calculated threshold factor was 1.3 [2]. Tables 1 and 2 summarize the stress in rebars at two critical locations. The maximum axial stress of 55.6 ksi is expected in rebars of the wall above east corner of Electrical Penetration.

6.3 Residual Heat Removal Equipment Vault

The stress in rebars of the critical components of RHR structure that governed the calculation of threshold factor is calculated and presented in Appendix C. Calculation of the threshold factor for the RHR structure is primarily governed by axial-flexure interaction along the horizontal strip along the south side of the east exterior wall [3]. A threshold factor of 1.2 was determined from evaluation of the RHR structure, which indicates that ASR-related demands are amplified by 20% beyond the factored values.

Tables 1 and 2 list the stress in horizontal rebars of east exterior wall, and the stress in vertical rebars in west and east interior walls of RHR structure. As can be seen from the table, the maximum tensile stress of 59.5 ksi is expected in the vertical rebars of the east interior wall due to LC2. However, the RHR walls are designed to span horizontally between intersecting walls; and therefore, the vertical rebars are not part of the main load path for the RHR. Figure C1 shows the contour plots of vertical strains in the interior walls due to LC1. The contour plots show that the overall vertical strains are reasonable compared to the yielding strain of rebars. Localized strain concentration is observed close to the door openings at approximate El. (-) 30 ft. and El. (-) 45 ft.

The next highest tensile stress is 56.5 ksi calculated for the horizontal rebars of exterior east wall. The specific section also governed the determination of threshold factor for the RHR structure. As explained in the susceptibility evaluation of RHR [3], moment can distribute to mid span and along the width of the wall, therefore, localized strain concentration is not of concern. The majority of the stresses that develop at this location are due to the RHR connection to PAB. The PAB foundation locally stiffens the connection between the RHR and the PAB which attracts the moment demand about the vertical axis in the east exterior wall of the RHR. In addition, the PAB base slab is subject to uplift pressure from backfill expansion which in turns induces forces in the RHR external walls near the connection. The stresses in the RHR evaluation and as reported here are conservative due to only including a limited model of PAB as connected to RHR which introduces extra overturning moment as well as the expected vertical shear force at this connection.

6.4 Condensate Storage Tank Enclosure

The stress in rebars of the critical components of CSTE structure that governed the calculation of threshold factor is calculated and presented in Appendix D. Selection of threshold factor for the CSTE structure is primarily governed by hoop tension at the top of the tank enclosure wall and vertical moment at the base of the tank enclosure wall [4]. A threshold factor of 1.6 was determined from evaluation of the CSTE structure, which indicates that ASR-related demands are amplified by 60% beyond the factored values.

Tables 1 and 2 summarize the stress in rebars of the tank enclosure wall of the CSTE structure. As can be seen from the table, the maximum axial stress of 26.7 ksi is expected to form in vertical rebars at the bottom of the tank enclosure wall.

6.5 Containment Equipment Hatch Missile Shield

The stress in rebars of the critical components of CEHMS structure that governed the calculation of threshold factor is calculated and presented in Appendix E. Selection of threshold factor for the CEHMS structure is primarily governed by out-of-plane moment at the base of east wing wall [5]. A threshold factor of 1.5 was determined from evaluation of the CEHMS structure, which indicates that ASR-related demands are amplified by 50% beyond the factored values.

Tables 1 and 2 summarize the stress in rebars of east wing wall of CEHMS structure. The maximum axial stress is 41.6 ksi expected to form in vertical rebars of the east wing wall at top of the column.

6.6 Containment Enclosure Ventilation Area

The stress in rebars of the critical components of CEVA structure that governed the calculation of threshold factor is calculated and presented in Appendix F. Selection of threshold factor for the CEVA is primarily governed by out-of-plane moment at the base slab located in Area 3 (Areas are defined in Ref. 6). A threshold factor of 3.0 was determined from evaluation of the

CEVA structure, which indicates that ASR-related demands are amplified by 200% beyond the factored values.

Tables 1 and 2 summarize the stress in rebars at the base slab. The maximum computed axial stress in rebars of the base mat is 44 ksi. However, as explained in Appendix F, the original design calculation did not provide demands due to unfactored load cases/combinations; hence, a conservative value was selected for the evaluation of rebar stress presented in Appendix F.

6.7 West Pipe Chase and Personnel Hatch

The stress in rebars at the critical flexural section of the WPC/PH structures is calculated and presented in Appendix I. The threshold factor of 1.8 was calculated based on out-of-plane shear of the WPC west wall [7]. Tables 1 and 2 summarize the stress in rebars at the base of the WPC north wall, the critical tensile stress location. A maximum tensile stress of 44.4 ksi develops in horizontal rebars of the WPC north wall.

6.8 Electrical Manholes

The stress in rebars at the critical flexural section of the EMH W13 and W15 is calculated and presented in Appendix G. The calculated threshold factor was 3.7 [8]. Tables 1 and 2 summarize the stress in rebars in EMH W13 and W15. A maximum tensile stress of 27.0 ksi develops in the horizontal rebars of EMH W13 and W15.

Table 1 – Stress in rebars of structural components subjected to LC1

	Component	Item		Internal ASR (mm/m)	Location	Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)	Maximum compressive mechanical strain in concrete
						Rebar 1	Rebar 2		
CRMAI	East Wall	M = 5.2	(kip-ft/ft)	0	East wall, horizontal strip, at the middle of the wall	36.2	26.8	0	>0
		P = 49.8	(kip/ft)						
	Base mat	M = 20.8	(kip-ft/ft)	0.99	North-south strip, at intersection with south walls	27.8	26.4	-0.28	-8.96e-5
		P = -28.4	(kip/ft)						
CEB[a]	Wall	M = 459.5	(kip-ft/ft)	0.60	Between Mechanical & Electrical Penetration at Elev. -30ft.	27.1	5.60	-2.21	-6.61e-4
		P = -141.2	(kip/ft)						
	Wall	M = -39.6	(kip-ft/ft)	0.10	Wall between Mechanical & Electrical Penetration, below personal hatch	24.6	2.73	-0.71	-1.88e-4
		P = 14.1	(kip/ft)						
RHR	East exterior wall	M = -98.5	(kip-ft/ft)	0.75	East exterior wall, horizontal strip, at the approximate El. (-) 30 ft	46.9	11.4	-1.9	-6.09e-4
		P = -35.0	(kip/ft)						
	East interior wall	M = 28.6	(kip-ft/ft)	0.0	East interior wall, vertical strip, at the approximate El. (-) 45 ft	41.6	5.5	0.0	>0
		P = 37.2	(kip/ft)						
	West interior wall	M = 11.0	(kip-ft/ft)	0.0	West interior wall, vertical strip, at the approximate El. (-) 30 ft	26.5	12.5	0.0	>0
		P = 30.8	(kip/ft)						

Table 1 – (Continue)

	Component	Item		Internal ASR (mm/m)	Location	Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)	Maximum compressive mechanical strain in concrete
						Rebar 1	Rebar 2		
CSTE	Tank Enclosure Wall	M = 41.0	(kip-ft/ft)	0.43	Bottom of tank enclosure wall, vertical direction	15.8	8.6	-0.68	-1.89e-4
		P = -12.9	(kip/ft)						
CEHMS	East wing walls	M = 159.6	(kip-ft/ft)	0.72	East wing wall, at intersection with column	23.4	15.0	-0.78	-2.50e-4
		P = -8.3	(kip/ft)						
CEVA	Base slab	M = 83.7	(kip-ft/ft)	0.31	Base slab rebar along east-west direction	32.8	5.1	-0.89	-2.8e-4
		P = 1.7	(kip/ft)						
WPC/PH[b]	North wall	M = 3.8	(kip-ft/ft)	0.24	North wall below pipe break beam	7.8	6.6	-0.07	-0.22e-4
		P = 19.1	(kip/ft)						
EMH[b]	W13/W15	M = 7.4	(kip-ft/ft)	0.25	W13/W15 walls	11.2	5.6	-0.28	-9.61e-6
		P = -3.2	(kip/ft)						

[a]Preliminary results, may change during checking and approval
[b]Calculation pending final review

Table 2 – Stress in rebars of structural components subjected to LC2

	Component	Item		F_{THR}	Internal ASR (mm/m)	Location	Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)	Maximum compressive mechanical strain in concrete
							Rebar 1	Rebar 2		
CRMAI	East Wall	M = 7.7	(kip-ft/ft)	1.4	0**	East wall, horizontal strip, at the middle of the wall	43.3	29.6	0	>0
		P = 57.6	(kip/ft)							
	Base mat	M = 26.5	(kip-ft/ft)		0.99	North-south strip, at intersection with south walls	39.1	37.3	-0.33	-1.06e-4
		P = -32.3	(kip/ft)							
CEB[a]	Wall	M = 614.7	(kip-ft/ft)	1.3	0.60	Between Mechanical & Electrical Penetration at Elev. -30ft.	42.5	1.97	-2.68	-8.51e-4
		P = 10.5	(kip/ft)							
	Wall	M =22.8	(kip-ft/ft)	1.3	0.10	East side of Electrical Penetration at Elev. 45ft.	55.6	12.9	-1.33	-3.67e-4
		P = 52.8	(kip/ft)							
RHR	East exterior wall	M = -119.5	(kip-ft/ft)	1.2	0.75	East exterior wall, horizontal strip, at the approximate El. (-) 30 ft	56.5	13.8	-2.1	-6.73e-4
		P = -40.8	(kip/ft)							
	East interior wall	M = 33.0	(kip-ft/ft)		0.0**	East interior wall, vertical strip, at the approximate El. (-) 45 ft	59.5*	17.7	0.0	>0
		P = 60.9	(kip/ft)							
	West interior wall	M = 13.4	(kip-ft/ft)		0.0**	West interior wall, vertical strip, at the approximate El. (-) 30 ft	36.6*	19.6	0.0	>0
		P = 44.4	(kip/ft)							

Table 2 – (Continued)

	Component	Item		F_{THR}	Internal ASR (mm/m)	Location	Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)	Maximum compressive mechanical strain in concrete
							Rebar 1	Rebar 2		
CSTE	Tank Enclosure Wall	M = 65.7	(kip-ft/ft)	1.6	0.43	Bottom of tank enclosure wall, vertical direction	26.7	13.9	-1.11	-3.08e-4
		P = -12.9	(kip/ft)							
CEHMS	East wing walls	M = 311.6	(kip-ft/ft)	1.5	0.72	East wing wall, at intersection with column	41.6	20.8	-1.52	-4.87e-4
		P = -0.7	(kip/ft)							
CEVA	Base slab	M = 83.7	(kip-ft/ft)	3.0	0.31	Base slab rebar along east-west direction	44.0	20.6	-1.08	-3.46e-4
		P = 1.7	(kip/ft)							
WPC/PH[b]	North wall	M = 78.8	(kip-ft/ft)	1.8	0.24	North wall below pipe break beam	44.4	8.0	-1.36	-4.37e-4
		P = 34.4	(kip/ft)							
EMH[b]	W13/W15	M = 0	(kip-ft/ft)	3.7	0.25	W13/W15 walls	27.0	24.5	-0.30	-9.69e-5
		P = 23.6	(kip/ft)							

* Vertical strips (strips that engage vertical rebars) are not part of primary load path for RHR, and therefore, are not designed following ACI 318 strength design method. These members do not need to be considered for the evaluation of stress in rebars.
** Members with zero internal ASR expansion that satisfy the ACI 318 requirements for strength design method do not yield subjected to unfactored normal operating load condition.
[a]Preliminary results, may change during checking and approval
[b]Calculation pending final review

7. REFERENCES

[1] Simpson Gumpertz & Heger Inc., *Evaluation of Control Room Makeup Air Intake Structure*, 160268-CA-08 Rev. 0, Waltham, MA, May 2017.

[2] Simpson Gumpertz & Heger, Inc., *Evaluation and Design Confirmation of As-designed CEB 150252-CA-02* Rev 1, Waltham, MA, Dec. 2017.

[3] Simpson Gumpertz & Heger Inc., *Evaluation of Residual Heat Removal Equipment Vault*, 160268-CA-06 Rev. 0, Waltham, MA, Dec 2016.

[4] Simpson Gumpertz & Heger Inc., *Evaluation of Condensate Storage Tank Enclosure Structure*, 160268-CA-03 Rev. 0, Waltham, MA, Dec. 2016.

[5] Simpson Gumpertz & Heger Inc., *Evaluation of Containment Equipment Hatch Missile Shield Structure*, 160268-CA-02 Rev. 0, Waltham, MA, Oct. 2016.

[6] Simpson Gumpertz & Heger Inc., *Evaluation of Containment Enclosure Ventilation Area*, 160268-CA-05 Rev. 0, Waltham, MA, Mar. 2017.

[7] Simpson Gumpertz & Heger, Inc., *Evaluation of Main Steam and Feedwater Pipe Chase & Personnel Hatch Structures 170443-CA-04* Rev. A, Waltham, MA, Nov. 2017.

[8] Simpson Gumpertz & Heger, Inc., *Evaluation of Seismic Category I Electrical Manholes – Stage 1 160268-CA-12* Rev. A, Waltham, MA, Nov. 2017.

APPENDIX A

TENSILE STRESS IN REBARS OF CONTROL ROOM MAKEUP AIR INTAKE STRUCTURE

A1. REVISION HISTORY

Revision 0: Initial document.

A2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the rebars of Control Room Makeup Air Intake (CRMAI) structure.

A3. RESULTS AND CONCLUSIONS

Table A1 summarizes the tensile stress in rebars of the CRMAI structure calculated at critical locations. The maximum tensile stress is 43.3 ksi computed for the horizontal rebar of east wall close to the interior of the structure and subjected to the second In Situ load combination.

Besides, although the stress due to internal ASR expansion is high for the base mat, the stress due to loading is small. Therefore, base mat does not govern the calculation of the maximum stress in rebars.

A4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 160268-CA-08 Rev. 0).

A5. ASSUMPTIONS

A5.1 Justified assumptions

There are no justified assumptions.

A5.2 Unverified assumptions

There are no unverified assumptions.

A6. METHODOLOGY

The critical demand that controlled the selection of threshold factor of the CRMAI structure was axial-flexure interaction along the horizontal strip of the east wall and close to the middle which is considered for evaluation. Additionally, the north-south strip of the base mat is also considered to check a location with high internal ASR expansion. Finite element analyses are conducted to calculate the axial force and bending moment at critical sections of the structure. The FE model and analysis method are similar to what explained in susceptibility evaluation of CRMAI structure [A1]. The axial force and bending moments are calculated using section cuts method. The computed demands are:

- LC1 for the walls: M = 5.2 kip-ft/ft, P = 49.8 kip/ft
- LC1 for the base mat: M = 20.8 kip-ft/ft, P = -28.4 kip/ft
- LC2 for the walls: M = 7.7 kip-ft/ft, P = 57.6 kip/ft
- LC2 for the base mat: M = 26.5 kip-ft/ft, P = -32.3 kip/ft

To calculate the stress in rebars subjected to a combination of axial force and bending moment, sectional analysis based on fiber section method, as explained in calculation main body, is used. The calculation is conducted per 1 foot width of the walls/slabs, and each section is discretized into 20 fibers. An example calculation that evaluates the stress in rebars of the east wall is presented in Section A8. The CI value for the base mat was 0.99 mm/m which included in the analysis to find the initial stress state due to internal ASR alone. Value of zero internal ASR is used for the walls as it leads to conservative demands.

A7. REFERENCES

[A1] Simpson Gumpertz & Heger Inc., *Evaluation of Control Room Makeup Air Intake structure,* 160268-CA-08 Rev. 0, Waltham, MA, May 2017.

[A2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[A3] United Engineers & Constructors Inc., *Design of Makeup Air Intake Structure*, *MT-28-Calc Rev.* 2, Feb. 1984.

A8. COMPUTATION

A8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index	$\varepsilon_{CI} := 0\dfrac{mm}{m}$	
Threshold factor	$F_{thr} := 1.4$	

Material properties

Compressive strength of concrete	$f_c := -3ksi$	Ref. [A1]
Young's modulus of concrete	$E_c := 3120ksi$	
Yield strength of steel	$f_y := 60ksi$	
Young's modulus of steel	$E_s := 29000ksi$	

Geometry

Width of fibers	$b := 12in$	Ref. [A2]
Total thickness or height	$h := 24in$	
Area of concrete	$A_c := b \cdot h = 288 \cdot in^2$	
Area of tensile reinforcement (#8@12 in.)	$A_s := 0.79in^2$	
Number of reinforcement in row, e.g. equal to 2 for tensile and compressive	$Steel_{Num} := 2$	
Depth to reinforcement	$d := 20.5in$	

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

	Initial Guess
Initial mechanical strain in concrete	$\varepsilon_{o.conc} := 0$
Initial strain in steel	$\varepsilon_{o.steel} := 0$
	Given
Compatibility equation	$F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$
Equilibrium equation	$\left(E_c \cdot A_c\right) \cdot \varepsilon_{o.conc} + \left(E_s \cdot A_s \cdot Steel_{Num}\right) \cdot \varepsilon_{o.steel} = 0$
	$ans := Find\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}\right)$

Initial strain in concrete and steel

$\varepsilon_{o.conc} := ans_1 = 0$

$\varepsilon_{o.steel} := ans_2 = 0$

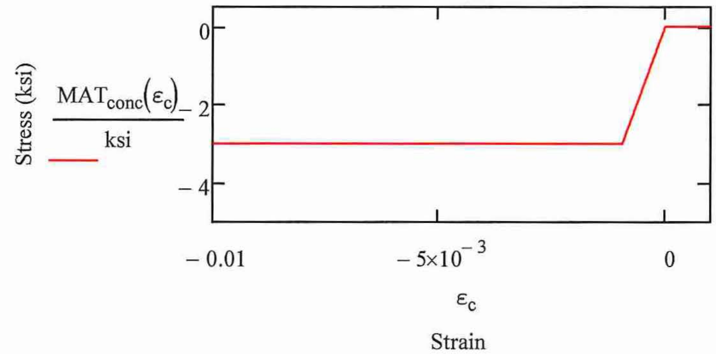
A8.2. Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

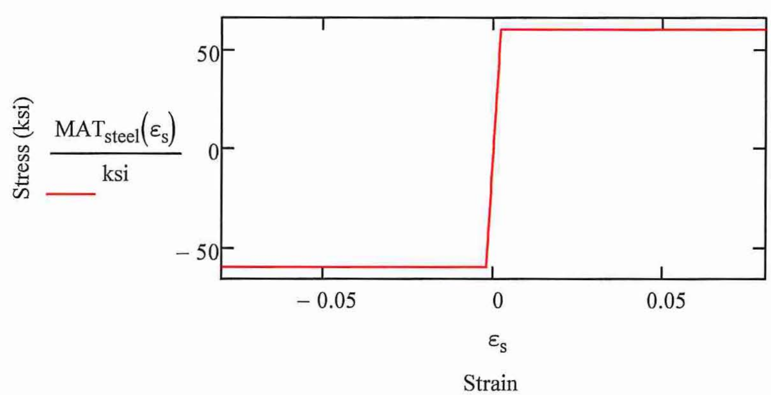
$$MAT_{conc}(\varepsilon) := \begin{cases} 0 & \text{if } \varepsilon > 0 \\ f_c & \text{if } \varepsilon < \dfrac{f_c}{E_c} \\ (E_c \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{cases} f_y & \text{if } \varepsilon > \dfrac{f_y}{E_s} \\ -f_y & \text{if } \varepsilon < \dfrac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Concrete Fibers

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \dfrac{h}{Conc_{Num}} = 1.2 \cdot in$

Concrete fiber coordinates

$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\dfrac{h}{2} + \dfrac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain

$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress

$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force

$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -8.5 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 8.5 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 0.79 \cdot in^2$$

$$Steel_{As_2} := A_s = 0.79 \cdot in^2$$

Steel fiber strain

$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress

$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force

$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$$\mathrm{Concrete}_\sigma := \mathrm{Conc}_\sigma(\varepsilon_{o.conc}, 0, 0)$$

$$\mathrm{Concrete}_{\sigma_1} = 0 \cdot \mathrm{ksi}$$

Initial stress in steel

$$\mathrm{Rebar}_\sigma := \mathrm{Steel}_\sigma(\varepsilon_{o.steel}, 0, 0)$$

$$\mathrm{Rebar}_{\sigma_1} = 0 \cdot \mathrm{ksi}$$

Axial Equilibrium

$$\mathrm{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \mathrm{ans1} \leftarrow 0 \\ \text{for } i \in 1..\mathrm{Conc_{Num}} \\ \quad \mathrm{ans1} \leftarrow \mathrm{ans1} + \mathrm{Conc_F}(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \\ \mathrm{ans2} \leftarrow 0 \\ \text{for } i \in 1..\mathrm{Steel_{Num}} \\ \quad \mathrm{ans2} \leftarrow \mathrm{ans2} + \mathrm{Steel_F}(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \\ \mathrm{ans} \leftarrow \mathrm{ans1} + \mathrm{ans2} \end{array} \right.$$

Moment Equilibrium

$$\mathrm{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \mathrm{ans1} \leftarrow 0 \\ \text{for } i \in 1..\mathrm{Conc_{Num}} \\ \quad \mathrm{ans1} \leftarrow \mathrm{ans1} + -1 \cdot \mathrm{Conc_F}(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot \mathrm{Conc}_{y_i} \\ \mathrm{ans2} \leftarrow 0 \\ \text{for } i \in 1..\mathrm{Steel_{Num}} \\ \quad \mathrm{ans2} \leftarrow \mathrm{ans2} + -1 \cdot \mathrm{Steel_F}(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot \mathrm{Steel}_{y_i} \\ \mathrm{ans} \leftarrow \mathrm{ans1} + \mathrm{ans2} \end{array} \right.$$

Solution

Known parameters

Axial force $P := 57.6\text{kip}$

Iteration

Curvature $\phi := 0.0000279 \cdot \frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium $f(x) := \text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi) - P$

$\varepsilon_{cent} := \text{root}(f(x_o), x_o) = 1.257 \times 10^{-3}$

Sectional forces

$\text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = 57.6 \cdot \text{kip}$

$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = 7.697 \cdot \text{kip} \cdot \text{ft}$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_\varepsilon := \text{Steel}_\varepsilon(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 1.494 \times 10^{-3} \\ 1.02 \times 10^{-3} \end{pmatrix}$$

$$\text{Rebar}_\sigma := \text{Steel}_\sigma(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 43.333 \\ 29.578 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_F(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 34.233 \\ 23.367 \end{pmatrix} \cdot \text{kip}$$

$\text{Concrete}_y := \text{Conc}_y$

Concrete fiber stress and strain

$\text{Concrete}_\varepsilon := \text{Conc}_\varepsilon(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$

$\text{Concrete}_\sigma := \text{Conc}_\sigma(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \ldots = 9.223 \times 10^{-4}$$
$$+ \text{Concrete}_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete $\sigma_{max.comp} := \text{MAT}_{conc}(\varepsilon_{max.comp}) = 0 \cdot \text{ksi}$

A9. TABLES

Table A1: Stress in rebars at critical locations of CRMAI structure subjected to LC1

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Walls	Out-of-plane moment (kip-ft/ft)	5.2	East wall, horizontal strip, at the middle of the wall	36.2	26.8	0
	Axial force (kip/ft)	49.8				
Base mat	Out-of-plane moment (kip-ft/ft)	20.8	North-south strip, at intersection with south walls	27.8	26.4	-0.28
	Axial force (kip/ft)	-28.4				

Table A2: Stress in rebars at critical locations of CRMAI structure subjected to LC2

Component	Item	Total demands for sustained loads plus OBE amplified with threshold factor (In Situ condition, LC2)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Walls	Out-of-plane moment (kip-ft/ft)	7.7	East wall, horizontal strip, at the middle of the wall	43.3	29.6	0
	Axial force (kip/ft)	57.6				
Base mat	Out-of-plane moment (kip-ft/ft)	26.5	North-south strip, at intersection with south walls	39.1	37.3	-0.33
	Axial force (kip/ft)	-32.3				

Example in Section A8

A10. FIGURES

There are no figures.

		PROJECT NO:	170444
		DATE:	Dec 2017
CLIENT:	NextEra Energy Seabrook	BY:	OOErbay
SUBJECT:	Evaluation of maximum stress in rebars of Seabrook structures	VERIFIER:	ATSarawit

APPENDIX B
TENSILE STRESS IN REBAR AND CONCRETE OF CONTAINMENT ENCLOSURE BUILDING STRUCTURE

B1. REVISION HISTORY

Revision 0: Initial document.

B2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the rebars and the maximum compressive stress that can form in concrete sections of the Containment Enclosure Building (CEB) structure.

B3. RESULTS AND CONCLUSIONS

Table B1 through B4 summarizes the stress results in rebar and concrete sections of the CEB structure calculated at critical locations. The Maximum tensile stress is 55.6 ksi in the wall at the east side of electrical penetration at Elev. 45 ft subjected to the second in-situ load combination (LC2).

B4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 150252-CA-02 Rev. 1).

B5. ASSUMPTIONS

B5.1 Justified assumptions

There are no justified assumptions.

B5.2 Unverified assumptions

There are no unverified assumptions.

B6. METHODOLOGY

The critical demands that control the selection of the threshold factor for the CEB structure are out-of-plane moment and axial load interaction at various sections of the wall surface. Finite element analyses were conducted to calculate the axial force and bending moment at these locations due to ASR load [B1].

To calculate the stress in rebars subjected to a combination of axial force and bending moment, sectional analysis based on fiber section method, as explained in calculation main body, is used. The calculation is conducted per 1 foot width of the walls, and each section is discretized into 20 fibers. An example calculation that evaluates the stress in the vertical rebars at the section of the wall on the east side of the electrical penetration and at Elev. 45 ft. is presented in Section B8. The ASR expansion of the CEB wall is included in the analysis to find the initial stress state due to internal ASR alone.

B7. REFERENCES

[B1] Simpson Gumpertz & Heger Inc., *Evaluation of Containment Enclosure Building Structure,* 150252-CA-02 Rev. 1, Waltham, MA, Dec 2017.

[B2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

B8. COMPUTATION

B8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index	$\varepsilon_{CI} := 0.10 \frac{mm}{m}$	
Threshold factor	$F_{thr} := 1.3$	

Material properties

Compressive strength of concrete	$f_c := -4ksi$	Ref. [B1]
Young's modulus of concrete	$E_c := 3605ksi$	
Yield strength of steel	$f_y := 60ksi$	
Young's modulus of steel	$E_s := 29000ksi$	

Geometry

Width of fibers	$b := 12in$	Ref. [B2]
Total thickness or height	$h := 15in$	
Area of concrete	$A_c := b \cdot h = 180 \cdot in^2$	
Area of tensile reinforcement	$A_s := 1.00in^2$	
Number of reinforcement in row, e.g. equal to 2 for tensile and compressive	$Steel_{Num} := 2$	
Depth to reinforcement	$d := 15in - 3.60in = 11.4 \cdot in$	

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

Initial Guess

Initial mechanical strain in concrete $\varepsilon_{o.conc} := 0$

Initial strain in steel $\varepsilon_{o.steel} := 0$

Given

Compatibility equation $F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$

Equilibrium equation $(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$

$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel $\varepsilon_{o.conc} := ans_1 = -1.217 \times 10^{-5}$

$\varepsilon_{o.steel} := ans_2 = 1.178 \times 10^{-4}$

B8.2. Sectional Analysis

Input Data

Concrete Material Model

Kent & Park Model

Strain at Peak compressive strength $\varepsilon_{co} := -0.002$

Strain at 50% compressive strength

$$\varepsilon_{50u} := \frac{3 - 0.002 \cdot \frac{f_c}{psi}}{\frac{f_c}{psi} + 1000} = -3.667 \times 10^{-3}$$

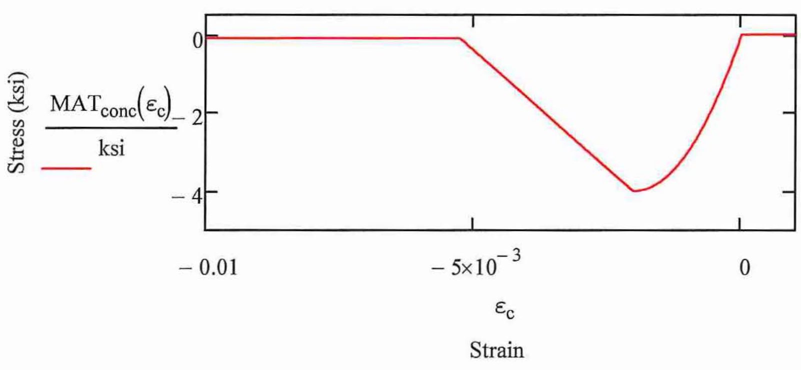
Model parameter

$$Z := \frac{0.5}{\varepsilon_{50u} - \varepsilon_{co}} = -300$$

Residual compressive strength $f_{c.res} := f_c \cdot 0.025 = -100 \cdot psi$

Constitutive model for concrete

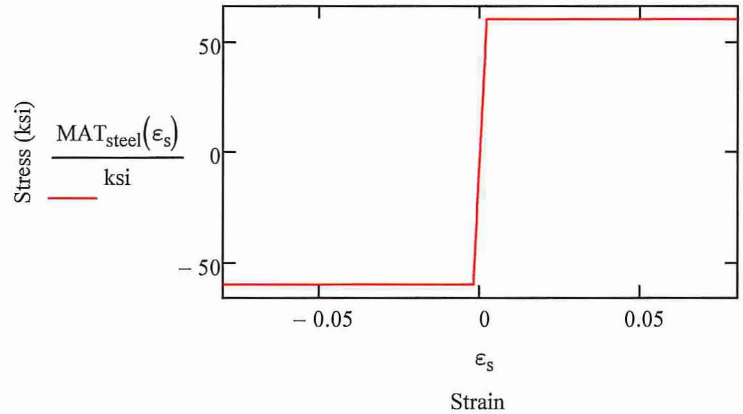
$$MAT_{conc}(\varepsilon) := \begin{cases} \min\left[f_{c.res}, f_c \cdot \left[1 - Z \cdot (\varepsilon - \varepsilon_{co})\right]\right] & \text{if } \varepsilon < \varepsilon_{co} \\ f_c \cdot \left[\frac{2 \cdot \varepsilon}{\varepsilon_{co}} - \left(\frac{\varepsilon}{\varepsilon_{co}}\right)^2\right] & \text{if } \varepsilon_{co} \le \varepsilon < 0 \\ 0 & \text{if } 0 \le \varepsilon \end{cases}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{array}{|l} f_y \ \text{ if } \varepsilon > \dfrac{f_y}{E_s} \\ -f_y \ \text{ if } \varepsilon < \dfrac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) \ \text{ otherwise} \end{array}$$



Concrete Fibers

Number of fibers

$$Conc_{Num} := 20$$

Height of fibers

$$Conc_H := \frac{h}{Conc_{Num}} = 0.75 \cdot in$$

Concrete fiber coordinates

$$Conc_y := \begin{array}{|l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow -\dfrac{h}{2} + \dfrac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array}$$

Concrete fiber strain

$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \begin{array}{|l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array}$$

Concrete fiber stress

$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \begin{array}{|l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array}$$

Concrete fiber force

$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \begin{array}{|l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array}$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -3.9 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 3.9 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 1 \cdot in^2$$

$$Steel_{As_2} := A_s = 1 \cdot in^2$$

Steel fiber strain

$$Steel_{\varepsilon}(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress

$$Steel_{\sigma}(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_{\varepsilon}(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force

$$Steel_{F}(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_{\sigma}(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$$Concrete_{\sigma} := Conc_{\sigma}(\varepsilon_{o.conc}, 0, 0)$$

$$Concrete_{\sigma_1} = -0.049 \cdot ksi$$

Initial stress in steel

$$Rebar_{\sigma} := Steel_{\sigma}(\varepsilon_{o.steel}, 0, 0)$$

$$Rebar_{\sigma_1} = 3.417 \cdot ksi$$

Axial Equilibrium

$$Force(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + Conc_{F}(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + Steel_{F}(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \\ ans \leftarrow ans1 + ans2 \end{array} \right.$$

Moment Equilibrium

$$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } i \in 1..\text{Conc}_{Num} \\ \quad \text{ans1} \leftarrow \text{ans1} + -1\cdot\text{Conc}_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\cdot\text{Conc}_{y_i} \\ \text{ans2} \leftarrow 0 \\ \text{for } i \in 1..\text{Steel}_{Num} \\ \quad \text{ans2} \leftarrow \text{ans2} + -1\cdot\text{Steel}_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\cdot\text{Steel}_{y_i} \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Solution

Known parameters

Axial force $P := 52.80\text{kip}$

Iteration

Curvature $\phi := 0.000189\cdot\frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium

$$f(x) := \text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi) - P$$

$$\varepsilon_{cent} := \text{root}(f(x_o), x_o) = 1.063 \times 10^{-3}$$

Sectional forces

$$\text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = 52.8\cdot\text{kip}$$

$$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = 22.807\cdot\text{kip}\cdot\text{ft}$$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_\varepsilon := \text{Steel}_\varepsilon(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 1.918 \times 10^{-3} \\ 4.434 \times 10^{-4} \end{pmatrix}$$

$$\text{Rebar}_\sigma := \text{Steel}_\sigma(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 55.61 \\ 12.858 \end{pmatrix}\cdot\text{ksi}$$

$$\text{Steel}_F(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 55.61 \\ 12.858 \end{pmatrix}\cdot\text{kip}$$

$$\text{Concrete}_y := \text{Conc}_y$$

Concrete fiber stress and strain

$$\text{Concrete}_\varepsilon := \text{Conc}_\varepsilon(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

$$\text{Concrete}_\sigma := \text{Conc}_\sigma(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{Concrete_{\varepsilon_{ConcNum}} - Concrete_{\varepsilon_{ConcNum-1}}}{Concy_{y_{ConcNum}} - Concy_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - Conc_{y_{ConcNum-1}}\right) \ldots = -3.67 \times 10^{-4}$$
$$+ Concrete_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete

$$\sigma_{max.comp} := MAT_{conc}(\varepsilon_{max.comp}) = -1.333 \cdot ksi$$

B9. TABLES

Table B1. Stress in Rebar and Concrete of Structural Components Subjected to LC1 Standard Case

Comp.	Demand		Location	Total stress in steel (ksi)		Maximum stress and strain in concrete (ksi) [in./in.]
				Rebar 1	Rebar 2	
Wall 36 in.	M = 459.5	(kip-ft/ft)	Wall near foundation. Horz. cut.	27.1	5.60	-2.21 [-6.61e-4]
	P = -141.2	(kip/ft)				
Wall 15 in.	M = 1.94	(kip-ft/ft)	Wall above Elec. Penetration. Horz. cut.	13.2	7.18	0.0 [2.25e-5]
	P = 20.33	(kip/ft)				
Wall 27 in.	M = -39.58	(kip-ft/ft)	Below personal hatch. Vert. cut.	24.6	2.73	-0.71 [-1.88e-4]
	P = 14.07	(kip/ft)				
Wall 27 in.	M = -34.00	(kip-ft/ft)	Side of personal hatch. Vert. cut.	19.5	2.78	-0.57 [-1.49e-4]
	P = 11.05	(kip/ft)				

Table B2. Stress in Rebar and Concrete of Structural Components subjected to LC2 Standard Case

Comp.	Demand		Location	Total stress in steel (ksi)		Maximum stress and strain in concrete (ksi) [in./in.]
				Rebar 1	Rebar 2	
Wall 36 in.	M = 614.7	(kip-ft/ft)	Wall near foundation. Horz. cut.	42.5	1.97	-2.68 [-8.507e-4]
	P = 10.48	(kip/ft)				
Wall 36 in.	M = 432.1	(kip-ft/ft)	Wall near foundation. Horz. cut.	20.6	2.16	-2.48 [-7.69e-4]
	P = -391.3	(kip/ft)				
Wall 15 in.	M =22.81	(kip-ft/ft)	Wall above Elec. Penetration. Horz. cut.	55.6	12.9	-1.33 [-3.67e-4]
	P = 52.80	(kip/ft)				
Wall 15 in.	M = -12.92	(kip-ft/ft)	Wall above Elec. Penetration. Horz. cut.	17.2	3.96	-0.78 [-2.042e-4]
	P = 4.70	(kip/ft)				
Wall 27 in.	M = -6.57	(kip-ft/ft)	Below personal hatch. Vert. cut.	25.9	19.7	0.0 [5.05e-4]
	P = 57.80	(kip/ft)				
Wall 27 in.	M = -92.23	(kip-ft/ft)	Below personal hatch. Vert. cut.	37.0	-1.17	-1.54 [-4.32e-4]
	P = -15.28	(kip/ft)				
Wall 27 in.	M = -1.18	(kip-ft/ft)	Side of personal hatch. Vert. cut.	22.5	21.4	0.0 [6.00e-4]
	P = 55.82	(kip/ft)				
Wall 27 in.	M = -80.76	(kip-ft/ft)	Side of personal hatch. Vert. cut.	27.8	-0.83	-1.29 [-3.55e-4]
	P = -21.38	(kip/ft)				

Table B3. Stress in Rebar and Concrete of Structural Components Subjected to LC1 Standard-Plus Case

Comp.	Demand		Location	Total stress in steel (ksi)		Maximum stress and strain in concrete (ksi) [in./in.]
				Rebar 1	Rebar 2	
Wall 36 in.	M = 459.2	(kip-ft/ft)	Wall near foundation. Horz. cut.	27.1	5.58	-2.21 [-6.61e-4]
	P = -142.2	(kip/ft)				
Wall 15 in.	M = 1.69	(kip-ft/ft)	Wall above Elec. Penetration. Horz. cut.	12.5	7.34	0.0 [4.03e-5]
	P = 19.88	(kip/ft)				
Wall 27 in.	M = -39.25	(kip-ft/ft)	Below personal hatch. Vert. cut.	24.6	2.73	-0.71 [-1.88e-4]
	P = 13.92	(kip/ft)				
Wall 27 in.	M = -33.66	(kip-ft/ft)	Side of personal hatch. Vert. cut.	19.3	2.79	-0.57 [-1.47e-4]
	P = 11.03	(kip/ft)				

Table B4. Stress in Rebar and Concrete of Structural Components Subjected to LC2 Standard-Plus Case

Comp.	Demand		Location	Total stress in steel (ksi)		Maximum stress and strain in concrete (ksi) [in./in.]
				Rebar 1	Rebar 2	
Wall 36 in.	M = 614.4	(kip-ft/ft)	Wall near foundation. Horz. cut.	42.4	1.97	-2.68 [-8.51e-4]
	P = 9.38	(kip/ft)				
Wall 36 in.	M = 431.8	(kip-ft/ft)	Wall near foundation. Horz. cut.	20.6	2.19	-2.48 [-7.67e-4]
	P = -392.3	(kip/ft)				
Wall 15 in.	M =22.45	(kip-ft/ft)	Wall above Elec. Penetration. Horz. cut.	54.9	12.8	-1.32 [-3.62e-4]
	P = 52.29	(kip/ft)				
Wall 15 in.	M = -13.08	(kip-ft/ft)	Wall above Elec. Penetration. Horz. cut.	17.1	3.90	-0.78 [-2.06e-4]
	P = 4.25	(kip/ft)				
Wall 27 in.	M = -6.24	(kip-ft/ft)	Below personal hatch. Vert. cut.	25.9	19.7	0.0 [5.05e-4]
	P = 57.62	(kip/ft)				
Wall 27 in.	M = -91.80	(kip-ft/ft)	Below personal hatch. Vert. cut.	37.0	-1.17	-1.54 [-4.32e-4]
	P = -15.46	(kip/ft)				
Wall 27 in.	M = -0.85	(kip-ft/ft)	Side of personal hatch. Vert. cut.	22.4	21.5	0.0 [6.07e-4]
	P = 55.76	(kip/ft)				
Wall 27 in.	M = -80.25	(kip-ft/ft)	Side of personal hatch. Vert. cut.	27.5	-0.81	-1.29 [-3.52e-4]
	P = -21.39	(kip/ft)				

B9. Figures

There are no figures

APPENDIX C

TENSILE STRESS IN REBARS OF RESIDUAL HEAT REMOVAL EQUIPMENT VAULT STRUCTURE

C1. REVISION HISTORY

Revision 0: Initial document.

C2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the reinforcing steel rebars of Residual Hear Removal Equipment Vault (RHR) structure.

C3. RESULTS AND CONCLUSIONS

Table C1 summarizes the tensile stress in rebars of the RHR structure calculated at critical locations. The maximum tensile stress is 59.5 ksi computed for the vertical rebar of east interior wall at approximate El. (-) 45 ft. and subjected to the second in situ load combination. However, per RHR susceptibility evaluation [C1] and original design calculation [C3], the vertical rebars are not the primary load path. Essentially, the wall were designed to span horizontally. The next highest stress value is 56.5 ksi that is computed for the east exterior wall.

C4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 160268-CA-06 Rev. 0).

C5. ASSUMPTIONS

C5.1 Justified assumptions

There are no justified assumptions.

C5.2 Unverified assumptions

There are no unverified assumptions.

C6. METHODOLOGY

The most critical stress demand in the horizontal rebars of the RHR structure is primarily due to the axial-flexure interaction along the vertical section cut in the south side of the east exterior wall. The highest stress demand in the vertical rebars of the RHR structure is primarily due to tension in the east and west interior walls.

Finite element analyses are conducted to calculate the axial force and bending moment at critical sections of the structure. The FE model and analysis method are similar to what explained in susceptibility evaluation of RHR structure [C1]. The axial force and bending moments are calculated using the method of section cuts.

Sectional analysis based on fiber section method is used to calculate the stress in the rebars of a section of a wall subjected to a combination of axial force and bending moment, as explained in calculation main body. Each wall section is discretized into 20 fibers of 1 ft width. An example calculation that evaluates the stress in the rebars of the east exterior wall is presented in Section C8. The CI value for the exterior wall was 0.75 mm/m which included in the analysis to find the initial stress state due to internal ASR alone. Zero internal ASR is used for the interior walls.

Figure C1 shows the contour plots of vertical strains in the interior walls due to LC1. The contour plots show that the overall vertical strains are reasonable compared to the yielding strain of rebars (i.e., 0.02% in/in). Localized strain concentration is observed close to the door openings at approximate El. (-) 30 ft. and El. (-) 45 ft.. Ductile distribution of local demands along the width of the interior walls is possible. As a result, localized strain concentration is not of concern.

C7. REFERENCES

[C1] Simpson Gumpertz & Heger Inc., *Evaluation of Residual Heat Removal Equipment Vault,* 160268-CA-06 Rev. 0, Waltham, MA, August 2017.

[C2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[C3] United Engineers & Constructors Inc., *Analysis and Design of Vault Walls up to El. 23 ft., PB-30 Calc Rev. 9*, Dec. 2002.

C8. COMPUTATION

C8.1 Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index $\varepsilon_{CI} := 0.75\frac{mm}{m}$

Threshold factor $F_{thr} := 1.0$

Material properties

Ref. [C1]

Compressive strength of concrete $f_c := -3ksi$

Young's modulus of concrete $E_c := 3120ksi$

Yield strength of steel $f_y := 60ksi$

Young's modulus of steel $E_s := 29000ksi$

Geometry

Ref. [C2]

Width of fibers $b := 12in$

Total thickness or height $h := 24in$

Area of concrete $A_c := b \cdot h = 288 \cdot in^2$

Area of tensile reinforcement (#8@9 in.) $A_s := 0.79 \cdot \frac{12}{9} in^2 = 1.053 \cdot in^2$

Number of reinforcement in row, e.g. equal to 2 for tensile and compressive $Steel_{Num} := 2$

Depth to reinforcement $d := 20.5in$

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

Initial Guess

Initial mechanical strain in concrete $\varepsilon_{o.conc} := 0$

Initial strain in steel $\varepsilon_{o.steel} := 0$

Given

Compatibility equation $F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$

Equilibrium equation $(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$

$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel

$\varepsilon_{o.conc} := ans_1 = -4.775 \times 10^{-5}$

$\varepsilon_{o.steel} := ans_2 = 7.023 \times 10^{-4}$

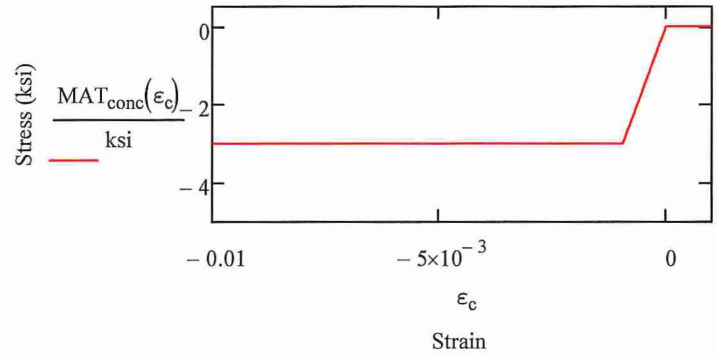
C8.2 Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

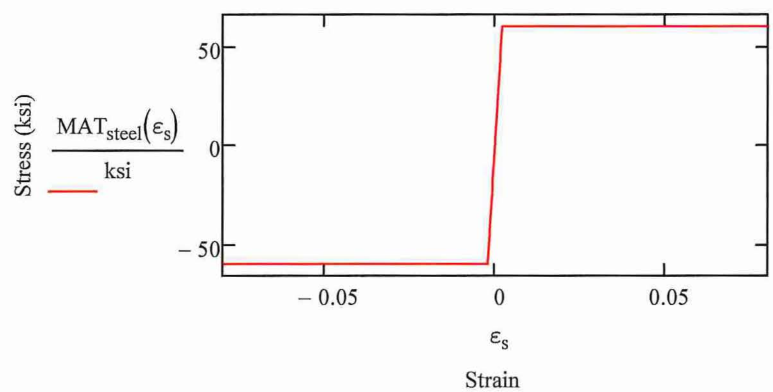
$$MAT_{conc}(\varepsilon) := \begin{cases} 0 & \text{if } \varepsilon > 0 \\ f_c & \text{if } \varepsilon < \frac{f_c}{E_c} \\ (E_c \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{cases} f_y & \text{if } \varepsilon > \frac{f_y}{E_s} \\ -f_y & \text{if } \varepsilon < \frac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Concrete Fibers

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \frac{h}{Conc_{Num}} = 1.2 \cdot in$

Concrete fiber coordinates
$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\frac{h}{2} + \frac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain
$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress
$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force
$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -8.5 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 8.5 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 1.053 \cdot in^2$$

$$Steel_{As_2} := A_s = 1.053 \cdot in^2$$

Steel fiber strain
$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress
$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force
$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$\text{Concrete}_{\sigma} := \text{Conc}_{\sigma}\left(\varepsilon_{o.conc}, 0, 0\right)$

$\text{Concrete}_{\sigma_1} = -0.149 \cdot \text{ksi}$

Initial stress in steel

$\text{Rebar}_{\sigma} := \text{Steel}_{\sigma}\left(\varepsilon_{o.steel}, 0, 0\right)$

$\text{Rebar}_{\sigma_1} = 20.365 \cdot \text{ksi}$

Axial Equilibrium

$$\text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } \ i \in 1 .. \text{Conc}_{\text{Num}} \\ \quad \text{ans1} \leftarrow \text{ans1} + \text{Conc}_{F}\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_{i} \\ \text{ans2} \leftarrow 0 \\ \text{for } \ i \in 1 .. \text{Steel}_{\text{Num}} \\ \quad \text{ans2} \leftarrow \text{ans2} + \text{Steel}_{F}\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_{i} \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Moment Equilibrium

$$\text{Moment}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } \ i \in 1 .. \text{Conc}_{\text{Num}} \\ \quad \text{ans1} \leftarrow \text{ans1} + -1 \cdot \text{Conc}_{F}\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_{i} \cdot \text{Conc}_{y_i} \\ \text{ans2} \leftarrow 0 \\ \text{for } \ i \in 1 .. \text{Steel}_{\text{Num}} \\ \quad \text{ans2} \leftarrow \text{ans2} + -1 \cdot \text{Steel}_{F}\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_{i} \cdot \text{Steel}_{y_i} \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Solution

Known parameters

Axial force $P := -35\text{kip}$

Iteration

Curvature $\phi := -0.000072 \cdot \frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium

$$f(x) := \text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi) - P$$

$$\varepsilon_{cent} := \text{root}(f(x_o), x_o) = 3.028 \times 10^{-4}$$

Sectional forces

$$\text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = -35 \cdot \text{kip}$$

$$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = -100.015 \cdot \text{kip} \cdot \text{ft}$$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_\varepsilon := \text{Steel}_\varepsilon(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 3.931 \times 10^{-4} \\ 1.617 \times 10^{-3} \end{pmatrix}$$

$$\text{Rebar}_\sigma := \text{Steel}_\sigma(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 11.399 \\ 46.895 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_F(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 12.007 \\ 49.396 \end{pmatrix} \cdot \text{kip}$$

$$\text{Concrete}_y := \text{Conc}_y$$

Concrete fiber stress and strain

$$\text{Concrete}_\varepsilon := \text{Conc}_\varepsilon(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

$$\text{Concrete}_\sigma := \text{Conc}_\sigma(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

Maximum compressive stress in concrete

$$\sigma_{max.comp} := \text{MAT}_{conc}\left(\text{Concrete}_{\varepsilon_1} + \phi \cdot \frac{\text{Conc}_H}{2}\right) = -1.9 \cdot \text{ksi}$$

C9. TABLES

Table C1: Stress in rebars at critical locations of RHR structure subjected to LC1

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Wall	Moment about the vertical global axis (kip-ft/ft)	-98.5	East exterior wall, vertical strip, at the approximate El. (-) 30 ft	46.9	11.4	-1.9
	Axial force (kip/ft)	-35.0				
Wall	Moment about the horizontal global axis (kip-ft/ft)	28.6	East interior wall, horizontal strip, at the approximate El. (-) 45 ft	41.6	5.5	0.0
	Axial force (kip/ft)	37.2				
Wall	Moment about the horizontal global axis (kip-ft/ft)	11.0	West interior wall, horizontal strip, at the approximate El. (-) 30 ft	26.5	12.5	0.0
	Axial force (kip/ft)	30.8				

Table C2: Stress in rebars at critical locations of RHR structure subjected to LC2

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Wall	Moment about the vertical global axis (kip-ft/ft)	-119.5	East exterior wall, vertical strip, at the approximate El. (-) 30 ft	56.5	13.8	-2.1
	Axial force (kip/ft)	-40.8				
Wall	Moment about the horizontal global axis (kip-ft/ft)	33.0	East interior wall, horizontal strip, at the approximate El. (-) 45 ft	59.5	17.7	0.0
	Axial force (kip/ft)	60.9				
Wall	Moment about the horizontal global axis (kip-ft/ft)	13.4	West interior wall, horizontal strip, at the approximate El. (-) 30 ft	36.6	19.6	0.0
	Axial force (kip/ft)	44.4				

C10. FIGURES

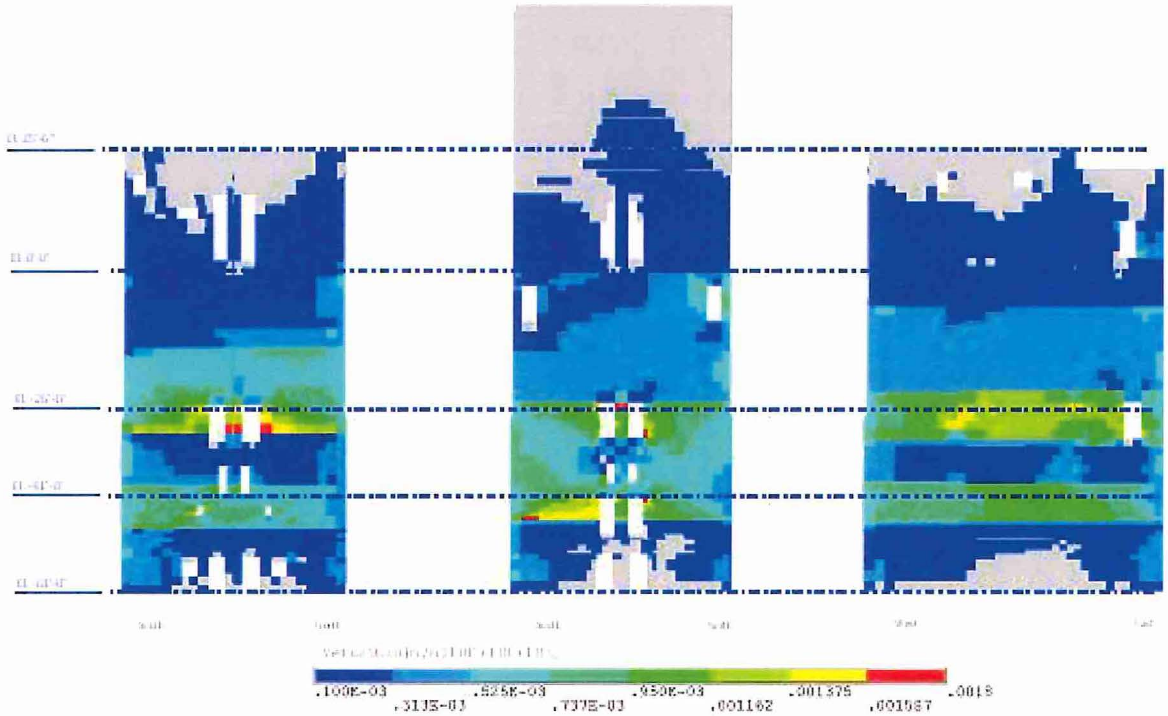


Figure C1: Contour plots of vertical strains in the interior walls due to LC1

APPENDIX D

TENSILE STRESS IN REBARS OF CONDENSATE STORAGE TANK ENCLOSURE STRUCTURE

D1. REVISION HISTORY

Revision 0: Initial document.

D2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the rebars of the Condensate Storage Tank Enclosure (CSTE) structure.

D3. RESULTS AND CONCLUSIONS

Table D1 summarizes the tensile stress in rebars of the CSTE structure calculated at critical locations. The Maximum tensile stress is 26.7 ksi at the bottom of the tank enclosure wall subjected to the second in situ load combination (LC2).

D4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 160268-CA-03 Rev. 0).

D5. ASSUMPTIONS

D5.1 Justified assumptions

There are no justified assumptions.

D5.2 Unverified assumptions

There are no unverified assumptions.

D6. METHODOLOGY

The critical demands that control the selection of the threshold factor for the CSTE structure are hoop tension at the top of the tank enclosure wall, and vertical moment at the base of the tank enclosure wall. Finite element analyses were conducted to calculate the axial force and bending moment at these locations due to ASR load [D1].

To calculate the stress in rebars subjected to a combination of axial force and bending moment, sectional analysis based on fiber section method, as explained in calculation main body, is used. The calculation is conducted per 1 foot width of the walls, and each section is discretized into 20 fibers. An example calculation that evaluates the stress in the vertical rebars at the base of the tank enclosure wall is presented in Section D8. The ASR expansion of the tank enclosure is included in the analysis to find the initial stress state due to internal ASR alone.

D7. REFERENCES

[D1] Simpson Gumpertz & Heger Inc., *Evaluation of Condensate Storage Tank Enclosure Structure,* 160268-CA-03 Rev. 0, Waltham, MA, Dec 2016.

[D2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[D3] United Engineers & Constructors Inc., *Condensate Storage Tank Mat and Wall Reinforcement*, MT-21, Rev. 3, Jan. 1984.

D8. COMPUTATION

D8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index	$\varepsilon_{CI} := 0.43 \frac{mm}{m}$	
Threshold factor	$F_{thr} := 1.6$	

Material properties

Compressive strength of concrete	$f_c := -4ksi$	Ref. [D1]
Young's modulus of concrete	$E_c := 3605ksi$	
Yield strength of steel	$f_y := 60ksi$	
Young's modulus of steel	$E_s := 29000ksi$	

Geometry

Width of fibers	$b := 12in$	Ref. [D2]
Total thickness or height	$h := 24in$	
Area of concrete	$A_c := b \cdot h = 288 \cdot in^2$	
Area of tensile reinforcement (#11@12 in.)	$A_s := 1.56in^2$	
Number of reinforcement in row, e.g. equal to 2 for tensile and compressive	$Steel_{Num} := 2$	
Depth to reinforcement	$d := 20.3in$	

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

	Initial Guess
Initial mechanical strain in concrete	$\varepsilon_{o.conc} := 0$
Initial strain in steel	$\varepsilon_{o.steel} := 0$
	Given
Compatibility equation	$F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$
Equilibrium equation	$(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$
	$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel

$$\varepsilon_{o.conc} := ans_1 = -5.515 \times 10^{-5}$$

$$\varepsilon_{o.steel} := ans_2 = 6.328 \times 10^{-4}$$

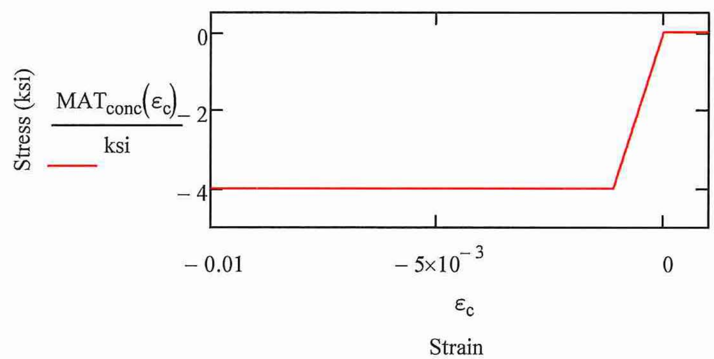
D8.2. Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

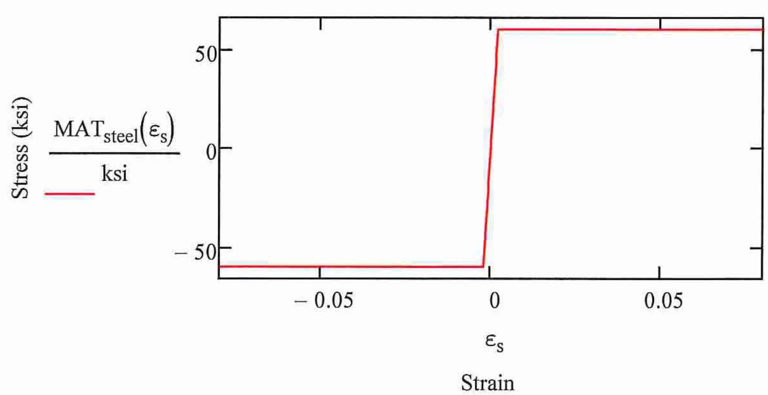
$$MAT_{conc}(\varepsilon) := \begin{cases} 0 & \text{if } \varepsilon > 0 \\ f_c & \text{if } \varepsilon < \dfrac{f_c}{E_c} \\ (E_c \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{cases} f_y & \text{if } \varepsilon > \dfrac{f_y}{E_s} \\ -f_y & \text{if } \varepsilon < \dfrac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Concrete Fibers

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \dfrac{h}{Conc_{Num}} = 1.2 \cdot in$

Concrete fiber coordinates

$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow -\dfrac{h}{2} + \dfrac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain

$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress

$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force

$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1 .. Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -8.3 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 8.3 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 1.56 \cdot in^2$$

$$Steel_{As_2} := A_s = 1.56 \cdot in^2$$

Steel fiber strain

$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1 .. Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress

$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1 .. Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force

$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1 .. Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$$Concrete_{\sigma} := Conc_{\sigma}\left(\varepsilon_{o.conc}, 0, 0\right)$$

$$Concrete_{\sigma_1} = -0.199 \cdot ksi$$

Initial stress in steel

$$Rebar_{\sigma} := Steel_{\sigma}\left(\varepsilon_{o.steel}, 0, 0\right)$$

$$Rebar_{\sigma_1} = 18.353 \cdot ksi$$

Axial Equilibrium

$$Force\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + Conc_F\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_i \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + Steel_F\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_i \\ ans \leftarrow ans1 + ans2 \end{array} \right.$$

Moment Equilibrium

$$Moment\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + -1 \cdot Conc_F\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_i \cdot Conc_{y_i} \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + -1 \cdot Steel_F\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_i \cdot Steel_{y_i} \\ ans \leftarrow ans1 + ans2 \end{array} \right.$$

Solution

Known parameters

Axial force

$P := -12.9\text{kip}$

Iteration

Curvature

$\phi := 0.0000266 \cdot \frac{1}{\text{in}}$

Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess)

$x_o := 0.0$

Axial force equilibrium

$f(x) := \text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi\right) - P$

$\varepsilon_{cent} := \text{root}\left(f\left(x_o\right), x_o\right) = 6.778 \times 10^{-5}$

Sectional forces

$\text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = -12.9 \cdot \text{kip}$

$\text{Moment}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = 65.634 \cdot \text{kip} \cdot \text{ft}$

Stress and strain in concrete and steel

Steel fiber stress and strain

$\text{Rebar}_{\varepsilon} := \text{Steel}_{\varepsilon}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 9.214 \times 10^{-4} \\ 4.799 \times 10^{-4} \end{pmatrix}$

$\text{Rebar}_{\sigma} := \text{Steel}_{\sigma}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 26.721 \\ 13.916 \end{pmatrix} \cdot \text{ksi}$

$\text{Steel}_{F}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 41.685 \\ 21.709 \end{pmatrix} \cdot \text{kip}$

$\text{Concrete}_{y} := \text{Conc}_{y}$

Concrete fiber stress and strain

$\text{Concrete}_{\varepsilon} := \text{Conc}_{\varepsilon}\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$

$\text{Concrete}_{\sigma} := \text{Conc}_{\sigma}\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \ldots + \text{Concrete}_{\varepsilon_{ConcNum-1}} = -3.066 \times 10^{-4}$$

Maximum compressive stress in concrete

$\sigma_{max.comp} := \text{MAT}_{conc}\left(\varepsilon_{max.comp}\right) = -1.105 \cdot \text{ksi}$

D9. TABLES

Table D1: Stress in rebars at critical locations of CSTE structure subjected to LC1

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Tank Enclosure Wall	Out-of-plane moment (kip-ft/ft)	0	Top of tank enclosure wall, horizontal direction	16.3	16.3	-0.14
	Axial force (kip/ft)	41.4				
	Out-of-plane moment (kip-ft/ft)	41	Bottom of tank enclosure wall, vertical direction	15.8	8.6	-0.68
	Axial force (kip/ft)	-12.9				

Table D2: Stress in rebars at critical locations of CSTE structure subjected to LC2

Component	Item	Total demands for sustained loads plus OBE amplified with threshold factor (In Situ condition, LC2)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Tank Enclosure Wall	Out-of-plane moment (kip-ft/ft)	0	Top of tank enclosure wall, horizontal direction	26.0	26.0	-0.23
	Axial force (kip/ft)	66.2				
	Out-of-plane moment (kip-ft/ft)	65.7	Bottom of tank enclosure wall, vertical direction	26.7	13.9	-1.11
	Axial force (kip/ft)	-12.9				

Example in Section D8

D10. FIGURES

There are no figures.

APPENDIX E

TENSILE STRESS IN REBARS OF CONTAINMENT EQUIPMENT HATCH MISSILE SHIELD STRUCTURE

E1. REVISION HISTORY

Revision 0: Initial document.

E2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the rebars of Containment Equipment Hatch Missile Shield (CEHMS) structure.

E3. RESULTS AND CONCLUSIONS

Table E1 summarizes the tensile stress in rebars of the CEHMS structure calculated at critical locations. The maximum tensile stress is 41.2 ksi computed for the eat wing wall at the intersection with the column.

E4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 160268-CA-02 Rev. 0).

E5. ASSUMPTIONS

E5.1 Justified assumptions

There are no justified assumptions.

E5.2 Unverified assumptions

There are no unverified assumptions.

E6. METHODOLOGY

The critical demand that governed the computation of the threshold factor of CEHMS structure was bending of east wind wall at the intersection with column. At this location the demands are:

- ASR load with threshold factor: M = 168 kip-ft/ft, P = 2.06 kip/ft (Appendix C of Ref. E1)
- Unfactored ASR load: M = 112.1 kip-ft/ft, P = 1.4 kip/ft (threshold factor was 1.5)
- Original unfactored demands excluding the OBE: M = 47.5 kip-ft/ft, P = -9.7 kip/ft (Sheet 30 to 45 of Ref. E3)
- Original unfactored demands including the OBE: M = 143.6 kip-ft/ft, P = -2.72 kip/ft (Sheet 30 to 45 of Ref. E3)

To calculate the stress in rebars subjected to a combination of axial force and bending moment, sectional analysis based on fiber section method, as explained in calculation main body, is used. The calculation is conducted per 1 foot width of the wall, and each section is discretized into 20 fibers. An example calculation that evaluates the stress in rebars of the east wing wall is presented in Section A8. The CI value of the wall was 0.72 mm/m which included in the analysis to find the initial stress state due to internal ASR alone.

E7. REFERENCES

[E1] Simpson Gumpertz & Heger Inc., *Evaluation of Containment Equipment Hatch Missile Shield structure,* 160268-CA-02 Rev. 0, Waltham, MA, Oct 2016.

[E2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[E3] United Engineers & Constructors Inc., *Equipment Hatch Shield Wall*, *CE-6-Calc Rev. 3*, Aug. 1998.

E8. COMPUTATION

E8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index	$\varepsilon_{CI} := 0.72 \frac{mm}{m}$
Threshold factor	$F_{thr} := 1.5$

Material properties

Compressive strength of concrete	$f_c := -3ksi$	Ref. [E1]
Young's modulus of concrete	$E_c := 3120ksi$	
Yield strength of steel	$f_y := 60ksi$	
Young's modulus of steel	$E_s := 29000ksi$	

Geometry

Width of fibers	$b := 12in$	Ref. [E2]
Total thickness or height	$h := 42in$	
Area of concrete	$A_c := b \cdot h = 504 \cdot in^2$	
Area of tensile reinforcement (#11@6 in.)	$A_s := 2 \cdot 1.56in^2$	
Number of reinforcement in row, e.g. equal to 2 for tensile and compressive	$Steel_{Num} := 2$	
Depth to reinforcement	$d := 36.88in$	

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

	Initial Guess
Initial mechanical strain in concrete	$\varepsilon_{o.conc} := 0$
Initial strain in steel	$\varepsilon_{o.steel} := 0$
	Given
Compatibility equation	$F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$
Equilibrium equation	$(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$
	$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel

$$\varepsilon_{o.conc} := ans_1 = -1.115 \times 10^{-4}$$

$$\varepsilon_{o.steel} := ans_2 = 9.685 \times 10^{-4}$$

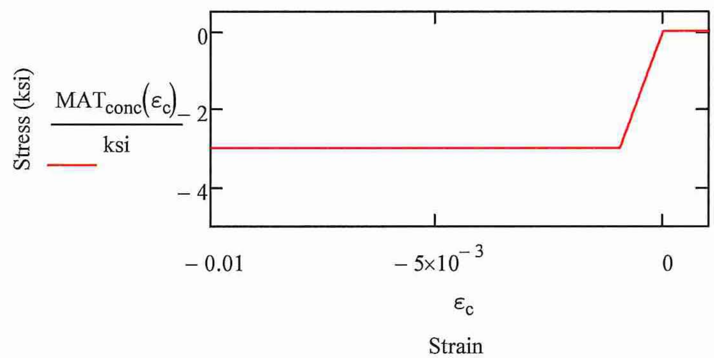
E8.2. Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

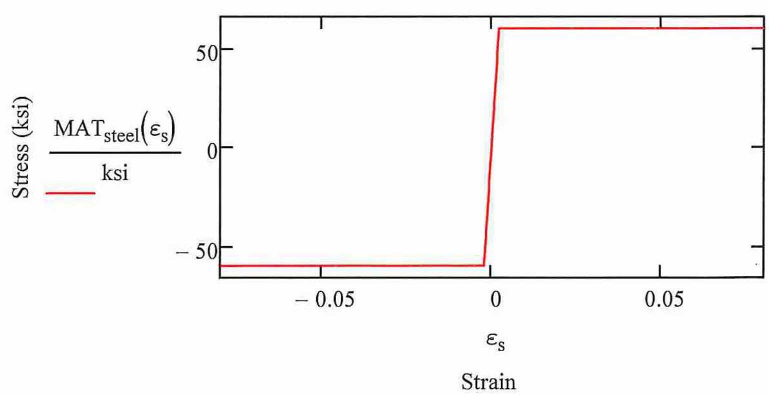
$$MAT_{conc}(\varepsilon) := \begin{vmatrix} 0 \text{ if } \varepsilon > 0 \\ f_c \text{ if } \varepsilon < \dfrac{f_c}{E_c} \\ (E_c \cdot \varepsilon) \text{ otherwise} \end{vmatrix}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{vmatrix} f_y \text{ if } \varepsilon > \dfrac{f_y}{E_s} \\ -f_y \text{ if } \varepsilon < \dfrac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) \text{ otherwise} \end{vmatrix}$$



Concrete Fibers

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \frac{h}{Conc_{Num}} = 2.1 \cdot in$

Concrete fiber coordinates
$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\frac{h}{2} + \frac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain
$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress
$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force
$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -15.88 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 15.88 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 3.12 \cdot in^2$$

$$Steel_{As_2} := A_s = 3.12 \cdot in^2$$

Steel fiber strain
$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress
$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force
$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$$Concrete_{\sigma} := Conc_{\sigma}(\varepsilon_{o.conc}, 0, 0)$$

$$Concrete_{\sigma_1} = -0.348 \cdot ksi$$

Initial stress in steel

$$Rebar_{\sigma} := Steel_{\sigma}(\varepsilon_{o.steel}, 0, 0)$$

$$Rebar_{\sigma_1} = 28.088 \cdot ksi$$

Axial Equilibrium

$$Force(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \\ ans \leftarrow ans1 + ans2 \end{array} \right.$$

Moment Equilibrium

$$Moment(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + -1 \cdot Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot Conc_{y_i} \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + -1 \cdot Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{y_i} \\ ans \leftarrow ans1 + ans2 \end{array} \right.$$

Solution

Known parameters

Axial force $P := -0.7\text{kip}$

Iteration

Curvature $\phi := 0.00002285 \cdot \frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium

$$f(x) := \text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi) - P$$

$$\varepsilon_{cent} := \text{root}(f(x_o), x_o) = 1.037 \times 10^{-4}$$

Sectional forces

$$\text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = -0.7 \cdot \text{kip}$$

$$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = 311.755 \cdot \text{kip} \cdot \text{ft}$$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_\varepsilon := \text{Steel}_\varepsilon(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 1.435 \times 10^{-3} \\ 7.094 \times 10^{-4} \end{pmatrix}$$

$$\text{Rebar}_\sigma := \text{Steel}_\sigma(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 41.618 \\ 20.572 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_F(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 129.848 \\ 64.186 \end{pmatrix} \cdot \text{kip}$$

$$\text{Concrete}_y := \text{Conc}_y$$

Concrete fiber stress and strain

$$\text{Concrete}_\varepsilon := \text{Conc}_\varepsilon(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

$$\text{Concrete}_\sigma := \text{Conc}_\sigma(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \ldots = -4.876 \times 10^{-4}$$
$$+ \text{Concrete}_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete $\sigma_{max.comp} := \text{MAT}_{conc}(\varepsilon_{max.comp}) = -1.521 \cdot \text{ksi}$

E9. TABLES

Table E1: Stress in rebars at critical locations of CEHMS structure subjected to LC1

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
East wing walls	Out-of-plane moment (kip-ft/ft)	159.6	East wing wall, at intersection with column	23.4	15.0	-0.78
	Axial force (kip/ft)	-8.3				

Table E2: Stress in rebars at critical locations of CEHMS structure subjected to LC2

Component	Item	Total demands for sustained loads plus OBE amplified with threshold factor (In Situ condition, LC2)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
East wing wall	Out-of-plane moment (kip-ft/ft)	311.6	East wall, horizontal strip, at the middle of the wall	41.6	20.8	-1.52
	Axial force (kip/ft)	-0.7				

Example in Section E8

E10. FIGURES

There are no figures.

APPENDIX F

TENSILE STRESS IN REBARS OF CONTAINMENT ENCLOSURE VENTILATION AREA

F1. REVISION HISTORY

Revision 0: Initial document.

F2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the rebars of Containment Enclosure Ventilation Area (CEVA) structure.

F3. RESULTS AND CONCLUSIONS

Table F1 summarizes the tensile stress in rebars of the CEVA structure calculated at critical locations. The maximum tensile stress is 44.0 ksi computed for the rebars of the base slab along east-west direction.

F4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 160268-CA-05 Rev. 0).

F5. ASSUMPTIONS

F5.1 Justified assumptions

There are no justified assumptions.

F5.2 Unverified assumptions

There are no unverified assumptions.

F6. METHODOLOGY

The critical demand that governed the computation of the threshold factor of CEVA structure was bending moment of the base slab in Area 3 subjected to seismic load combinations that act parallel to east-west direction [F1]. The original calculation of CEVA structure [F3] does not provide unfactored demand values; therefore, in this evaluation, the factored load is conservatively divided by the minimum load factor in the load combination and used in calculating rebar stress:

- ASR load with threshold factor: M = 28.7 kip-ft/ft P = 0 (Appendix C of Ref. F1)
- Unfactored ASR load: M = 9.56 kip-ft/ft, P = 0 (threshold factor was 3.0)
- Original unfactored demands incxluding the OBE: M = 77/1.4 = 55 kip-ft/ft, P = 2.44/1.4 = 1.43 kip/ft (Sheet 16 of Ref. F3). Note that the value of 1.4 was the load factor applied to the dead load in the combination (minimum load factor)

To calculate the stress in rebars subjected to a combination of axial force and bending moment, sectional analysis based on fiber section method, as explained in calculation main body, is used. The calculation is conducted per 1 foot width, and each section is discretized into 20 fibers. An example calculation that evaluates the stress in rebars of the base slab is presented in Section F8. The CI value of all components was set equal to 0.31 mm/m which included in the analysis to find the initial stress state due to internal ASR alone.

F7. REFERENCES

[F1] Simpson Gumpertz & Heger Inc., *Evaluation of Containment Enclosure Ventilation Area,* 160268-CA-05 Rev. 0, Waltham, MA, Mar. 2017.

[F2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[F3] United Engineers & Constructors Inc., *Containment Enclosure Ventilation Area*, *EM-33-Calc Rev. 4*, Jan. 1986.

F8. COMPUTATION

F8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index $\varepsilon_{CI} := 0.31 \frac{mm}{m}$

Threshold factor $F_{thr} := 3$

Material properties

Compressive strength of concrete $f_c := -3ksi$ Ref. [F1]

Young's modulus of concrete $E_c := 3120ksi$

Yield strength of steel $f_y := 60ksi$

Young's modulus of steel $E_s := 29000ksi$

Geometry

Width of fibers $b := 12in$ Ref. [F2]

Total thickness or height $h := 30in$

Area of concrete $A_c := b \cdot h = 360 \cdot in^2$

Area of tensile reinforcement (#9@12 in.) $A_s := 1in^2$

Number of reinforcement in row, e.g. equal to 2 for tensile and compressive $Steel_{Num} := 2$

Depth to reinforcement $d := 26.4in$

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

Initial Guess

Initial mechanical strain in concrete $\varepsilon_{o.conc} := 0$

Initial strain in steel $\varepsilon_{o.steel} := 0$

Given

Compatibility equation $F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$

Equilibrium equation $(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$

$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel

$\varepsilon_{o.conc} := ans_1 = -4.567 \times 10^{-5}$

$\varepsilon_{o.steel} := ans_2 = 8.843 \times 10^{-4}$

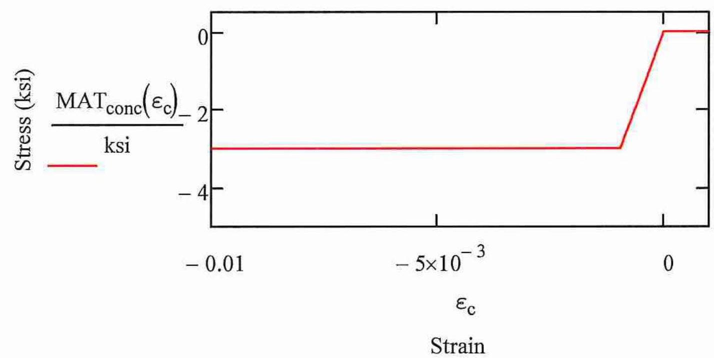
F8.2. Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

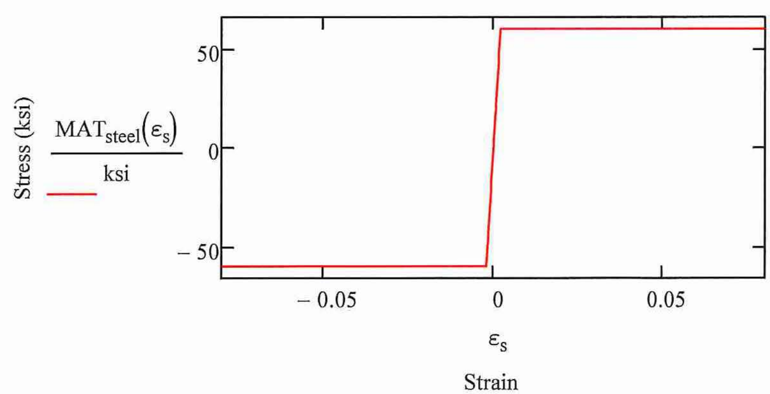
$$MAT_{conc}(\varepsilon) := \begin{cases} 0 & \text{if } \varepsilon > 0 \\ f_c & \text{if } \varepsilon < \dfrac{f_c}{E_c} \\ (E_c \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{cases} f_y & \text{if } \varepsilon > \dfrac{f_y}{E_s} \\ -f_y & \text{if } \varepsilon < \dfrac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) & \text{otherwise} \end{cases}$$



Concrete Fibers

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \dfrac{h}{Conc_{Num}} = 1.5 \cdot in$

Concrete fiber coordinates

$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\dfrac{h}{2} + \dfrac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain

$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress

$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force

$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -11.4 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 11.4 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 1 \cdot in^2$$

$$Steel_{As_2} := A_s = 1 \cdot in^2$$

Steel fiber strain

$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress

$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force

$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$Concrete_{\sigma} := Conc_{\sigma}(\varepsilon_{o.conc}, 0, 0)$

$Concrete_{\sigma_1} = -0.142 \cdot ksi$

Initial stress in steel

$Rebar_{\sigma} := Steel_{\sigma}(\varepsilon_{o.steel}, 0, 0)$

$Rebar_{\sigma_1} = 25.646 \cdot ksi$

Axial Equilibrium

$$
Force(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left|
\begin{array}{l}
ans1 \leftarrow 0 \\
\text{for } i \in 1 .. Conc_{Num} \\
\quad ans1 \leftarrow ans1 + Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \\
ans2 \leftarrow 0 \\
\text{for } i \in 1 .. Steel_{Num} \\
\quad ans2 \leftarrow ans2 + Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \\
ans \leftarrow ans1 + ans2
\end{array}
\right.
$$

Moment Equilibrium

$$
Moment(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left|
\begin{array}{l}
ans1 \leftarrow 0 \\
\text{for } i \in 1 .. Conc_{Num} \\
\quad ans1 \leftarrow ans1 + -1 \cdot Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot Conc_{y_i} \\
ans2 \leftarrow 0 \\
\text{for } i \in 1 .. Steel_{Num} \\
\quad ans2 \leftarrow ans2 + -1 \cdot Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{y_i} \\
ans \leftarrow ans1 + ans2
\end{array}
\right.
$$

Solution

Known parameters

Axial force $P := 1.74\text{kip}$

Iteration

Curvature $\phi := 0.0000353 \cdot \frac{1}{in}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium

$$f(x) := \text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi\right) - P$$

$$\varepsilon_{cent} := \text{root}\left(f\left(x_o\right), x_o\right) = 2.299 \times 10^{-4}$$

Sectional forces

$$\text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = 1.74 \cdot \text{kip}$$

$$\text{Moment}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = 83.675 \cdot \text{kip} \cdot \text{ft}$$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_{\varepsilon} := \text{Steel}_{\varepsilon}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 1.517 \times 10^{-3} \\ 7.118 \times 10^{-4} \end{pmatrix}$$

$$\text{Rebar}_{\sigma} := \text{Steel}_{\sigma}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 43.982 \\ 20.642 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_{F}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 43.982 \\ 20.642 \end{pmatrix} \cdot \text{kip}$$

$$\text{Concrete}_{y} := \text{Conc}_{y}$$

Concrete fiber stress and strain

$$\text{Concrete}_{\varepsilon} := \text{Conc}_{\varepsilon}\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$$

$$\text{Concrete}_{\sigma} := \text{Conc}_{\sigma}\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \ldots = -3.453 \times 10^{-4}$$
$$+ \text{Concrete}_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete $\sigma_{max.comp} := \text{MAT}_{conc}\left(\varepsilon_{max.comp}\right) = -1.077 \cdot \text{ksi}$

F9. TABLES

Table F1: Stress in rebars at critical locations of CEVA structure subjected to LC1

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Base slab	Out-of-plane moment (kip-ft/ft)	64.5*	Base slab at Area 3	32.8	5.1	-0.89
	Axial force (kip/ft)	1.74*				

These demands are computed conservatively by including OBE and dividing the total factor demand by the minimum load factor in the load combination in the original design calculation.

Table F2: Stress in rebars at critical locations of CEVA structure subjected to LC2

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
Base slab	Out-of-plane moment (kip-ft/ft)	83.7	Base slab at Area 3	44.0	20.6	-1.08
	Axial force (kip/ft)	1.74				

Example in Section F8

F10. FIGURES

There are no figures.

APPENDIX G

TENSILE STRESS IN REBARS OF STAGE 1 ELECTRICAL MANHOLES

G1. REVISION HISTORY

Revision 0: Initial document.

G2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the rebars of the Stage 1 Electrical Manhole (EMH) structures.

G3. RESULTS AND CONCLUSIONS

Table G1 summarizes the tensile stress in rebars of the EMH calculated at critical locations. The maximum tensile stress is 27 ksi in EMH W13/W15 subjected to the second in situ load combination (LC2).

G4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 160268-CA-12 Rev. A).

G5. ASSUMPTIONS

G5.1 Justified assumptions

There are no justified assumptions.

G5.2 Unverified assumptions

There are no unverified assumptions.

G6. METHODOLOGY

The critical demands that control rebar tension in the Stage 1 EMH are horizontal moment and horizontal tension in EMH W13 and W15. Finite element analyses were conducted to calculate the axial force and bending moment at these locations due to ASR load [G1].

To calculate the stress in rebars subjected to a combination of axial force and bending moment, sectional analysis based on fiber section method, as explained in calculation main body, is used. The calculation is conducted per 1 foot width of the walls, and each section is discretized into 20 fibers. An example calculation that evaluates the stress in the horizontal rebars in the walls of EMH W13 and W15 is presented in Section G8. The ASR expansion of the EMH is included in the analysis to find the initial stress state due to internal ASR alone.

G7. REFERENCES

[G1] Simpson Gumpertz & Heger Inc., *Evaluation of Seismic Category I Electrical Manholes – Stage 1,* 160268-CA-12 Rev. A, Waltham, MA, Nov 2017.

[G2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

G8. COMPUTATION

G8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index	$\varepsilon_{CI} := 0.25 \dfrac{mm}{m}$
Threshold factor	$F_{thr} := 3.7$

Material properties

Compressive strength of concrete	$f_c := -3ksi$	Ref. [G1]
Young's modulus of concrete	$E_c := 3120ksi$	
Yield strength of steel	$f_y := 60ksi$	
Young's modulus of steel	$E_s := 29000ksi$	

Geometry

Width of fibers	$b := 12in$	Ref. [G2]
Total thickness or height	$h := 18in$	
Area of concrete	$A_c := b \cdot h = 216 \cdot in^2$	
Area of tensile reinforcement (#6@12 in.)	$A_s := 0.44in^2$	
Number of reinforcement in row, e.g. equal to 2 for tensile and compressive	$Steel_{Num} := 2$	
Depth to reinforcement	$d := 15.625in$	

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

	Initial Guess
Initial mechanical strain in concrete	$\varepsilon_{o.conc} := 0$
Initial strain in steel	$\varepsilon_{o.steel} := 0$
	Given
Compatibility equation	$F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$
Equilibrium equation	$(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$
	$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel

$$\varepsilon_{o.conc} := ans_1 = -3.375 \times 10^{-5}$$

$$\varepsilon_{o.steel} := ans_2 = 8.913 \times 10^{-4}$$

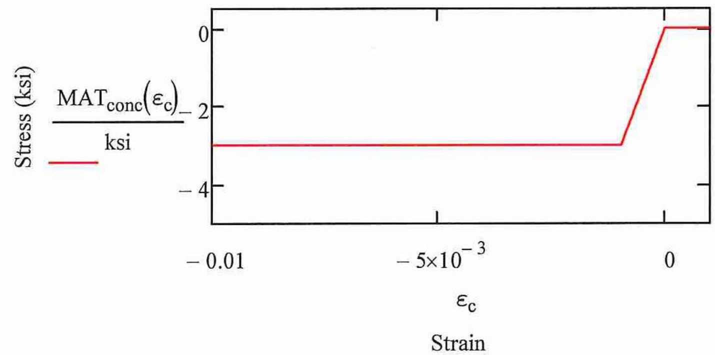
G8.2. Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

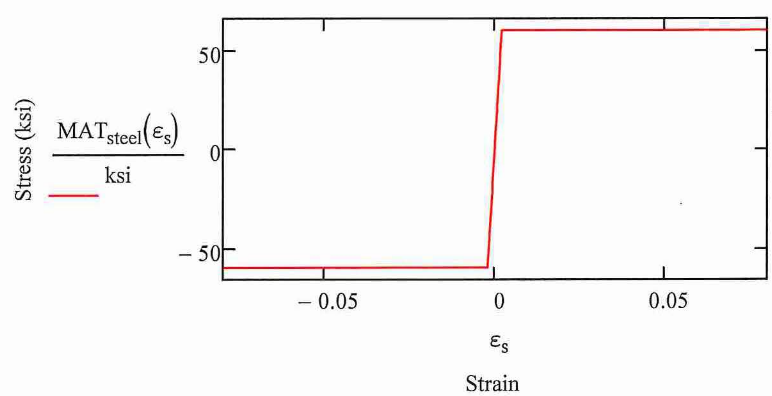
$$MAT_{conc}(\varepsilon) := \begin{array}{|l} 0 \ \text{ if } \varepsilon > 0 \\ f_c \ \text{ if } \varepsilon < \dfrac{f_c}{E_c} \\ (E_c \cdot \varepsilon) \ \text{ otherwise} \end{array}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{array}{|l} f_y \ \text{ if } \varepsilon > \dfrac{f_y}{E_s} \\ -f_y \ \text{ if } \varepsilon < \dfrac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) \ \text{ otherwise} \end{array}$$



Concrete Fibers

Number of fibers $\quad Conc_{Num} := 20$

Height of fibers $\quad Conc_H := \dfrac{h}{Conc_{Num}} = 0.9 \cdot in$

Concrete fiber coordinates

$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\dfrac{h}{2} + \dfrac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain

$$Conc_{\varepsilon}(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress

$$Conc_{\sigma}(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_{\varepsilon}(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force

$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_{\sigma}(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -6.625 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 6.625 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 0.44 \cdot in^2$$

$$Steel_{As_2} := A_s = 0.44 \cdot in^2$$

Steel fiber strain

$$Steel_{\varepsilon}(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress

$$Steel_{\sigma}(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_{\varepsilon}(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force

$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_{\sigma}(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$$\text{Concrete}_\sigma := \text{Conc}_\sigma(\varepsilon_{o.conc}, 0, 0)$$

$$\text{Concrete}_{\sigma_1} = -0.105 \cdot \text{ksi}$$

Initial stress in steel

$$\text{Rebar}_\sigma := \text{Steel}_\sigma(\varepsilon_{o.steel}, 0, 0)$$

$$\text{Rebar}_{\sigma_1} = 25.846 \cdot \text{ksi}$$

Axial Equilibrium

$$\text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Conc}_{Num} \\ \quad \text{ans1} \leftarrow \text{ans1} + \text{Conc}_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \\ \text{ans2} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Steel}_{Num} \\ \quad \text{ans2} \leftarrow \text{ans2} + \text{Steel}_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Moment Equilibrium

$$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Conc}_{Num} \\ \quad \text{ans1} \leftarrow \text{ans1} + -1 \cdot \text{Conc}_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot \text{Conc}_{y_i} \\ \text{ans2} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Steel}_{Num} \\ \quad \text{ans2} \leftarrow \text{ans2} + -1 \cdot \text{Steel}_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot \text{Steel}_{y_i} \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Solution

Known parameters

Axial force $P := -4.4\text{kip}$

Iteration

Curvature $\phi := 0.0000065 \cdot \frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium $f(x) := \text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi) - P$

$\varepsilon_{cent} := \text{root}(f(x_o), x_o) = -4.655 \times 10^{-6}$

Sectional forces

$\text{Force}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = -4.4 \cdot \text{kip}$

$\text{Moment}(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = 9.679 \cdot \text{kip} \cdot \text{ft}$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_{\varepsilon} := \text{Steel}_{\varepsilon}(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 9.297 \times 10^{-4} \\ 8.435 \times 10^{-4} \end{pmatrix}$$

$$\text{Rebar}_{\sigma} := \text{Steel}_{\sigma}(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 26.96 \\ 24.462 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_{F}(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi) = \begin{pmatrix} 11.862 \\ 10.763 \end{pmatrix} \cdot \text{kip}$$

$\text{Concrete}_y := \text{Conc}_y$

Concrete fiber stress and strain

$\text{Concrete}_{\varepsilon} := \text{Conc}_{\varepsilon}(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$

$\text{Concrete}_{\sigma} := \text{Conc}_{\sigma}(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi)$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \dots = -9.69 \times 10^{-5}$$
$$+ \text{Concrete}_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete $\sigma_{max.comp} := \text{MAT}_{conc}(\varepsilon_{max.comp}) = -0.302 \cdot \text{ksi}$

G9. TABLES

Table G1: Stress in rebars at critical locations of EMH subjected to LC1

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
EMH W13/W15	Out-of-plane moment (kip-ft/ft)	7.4	W13/W15 wall	11.2	5.6	-0.28
	Axial force (kip/ft)	-3.2				

Table G2: Stress in rebars at critical locations of EMH subjected to LC2

Component	Item	Total demands for sustained loads plus OBE amplified with threshold factor (In Situ condition, LC2)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
EMH W13/W15	Out-of-plane moment (kip-ft/ft)	9.3	W13/W15 wall	27.0	24.5	-0.30
	Axial force (kip/ft)	-4.4				

Example in Section G8

G10. FIGURES

There are no figures.

APPENDIX H

EVALUATING THE PERFORMANCE OF A SIMPLE ELASTO-PLASTIC MATERIAL MODEL FOR CONCRETE TO BE USED FOR EVALUATION OF REBAR STRESS

H1. REVISION HISTORY

Revision 0: Initial document.

H2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compare the rebar stresses computed by using two different using constitutive models for concrete, and justify the at the simple material model provides a satisfactory results.

H3. RESULTS AND CONCLUSIONS

The stress in rebars are computed using two constitutive models for concrete. The stress in rebars obtained using both models are very close indicating the simple model captures the concrete behavior satisfactorily. This is due to steel ratios in the components of Seabrook structures which is less than the maximum ratio allowed by the code. Therefore concrete crushing and post-linear response of the concrete does not impact the response noticeably.

CRMAI:

- Stress in Rebar 1: 39.1 (simple model, Appendix A) and 39.01 (accurate model)
- Stress in Rebar 2: 37.3 (simple model, Appendix A) and 37.16 (accurate model)
- Stress in concrete: -0.33 (simple model, Appendix A) and -0.328 (accurate model)

CEHMS:

- Stress in Rebar 1: 41.6 (simple model, Appendix E) and 42.1 (accurate model)
- Stress in Rebar 2: 20.8 (simple model, Appendix E) and 19.3 (accurate model)
- Stress in concrete: -1.5 (simple model, Appendix E) and -1.4 (accurate model)

H4. DESIGN DATA / CRITERIA

There are no design data.

H5. ASSUMPTIONS

H5.1 Justified assumptions

There are no justified assumptions.

H5.2 Unverified assumptions

There are no unverified assumptions.

H6. METHODOLOGY

Stress in rebars at the base mat of CRMAI and at the east wing wall of CEHMS structures are computed using a more accurate constitutive model of Kent and Park [H4] in compression, and the results are compared with the stresses obtained from the simple model as explained in the main body. The stresses in rebars from the simple model are provided in Appendix A and E for CRMAI and CEHMS structures respectively. Section H8 provides a sample calculation for rhe base mat of CRMAI structures.

H7. REFERENCES

[H1] Simpson Gumpertz & Heger Inc., *Evaluation of Control Room Makeup Air Intake structure,* 160268-CA-08 Rev. 0, Waltham, MA, May 2017.

[H2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[H3] United Engineers & Constructors Inc., *Design of Makeup Air Intake Structure*, *MT-28-Calc Rev. 2*, Feb. 1984.

[H4] Dudley. C. Kent, and Robert Park, Flexural members with confined concrete, ASCE Journal of Structural Division, 97 (ST7), 1969-1990, 1971.

H8. COMPUTATION

H8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index	$\varepsilon_{CI} := 0.99\frac{mm}{m}$
Threshold factor	$F_{thr} := 1.4$

Material properties

Compressive strength of concrete	$f_c := -3ksi$	Ref. [H1]
Young's modulus of concrete	$E_c := 3120ksi$	

Yield strength of steel $f_y := 60ksi$

Young's modulus of steel $E_s := 29000ksi$

Geometry

Width of fibers $b := 12in$ Ref. [H2]

Total thickness or height $h := 36in$

Area of concrete $A_c := b \cdot h = 432 \cdot in^2$

Area of tensile reinforcement (#8@12 in.) $A_s := 0.79in^2$

Number of reinforcement in row, e.g. equal to 2 for tensile and compressive $Steel_{Num} := 2$

Depth to reinforcement $d := 32.5in$

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

Initial Guess

Initial mechanical strain in concrete $\varepsilon_{o.conc} := 0$

Initial strain in steel $\varepsilon_{o.steel} := 0$

Given

Compatibility equation $F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$

Equilibrium equation $(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$

$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel

$\varepsilon_{o.conc} := ans_1 = -4.557 \times 10^{-5}$

$\varepsilon_{o.steel} := ans_2 = 1.34 \times 10^{-3}$

H8.2. Sectional Analysis

Input Data

Concrete Material Model

Kent & Park Model

Strain at Peak compressive strength $\varepsilon_{co} := -0.002$

Strain at 50% compressive strength

$$\varepsilon_{50u} := \frac{3 - 0.002 \cdot \frac{f_c}{psi}}{\frac{f_c}{psi} + 1000} = -4.5 \times 10^{-3}$$

Model parameter

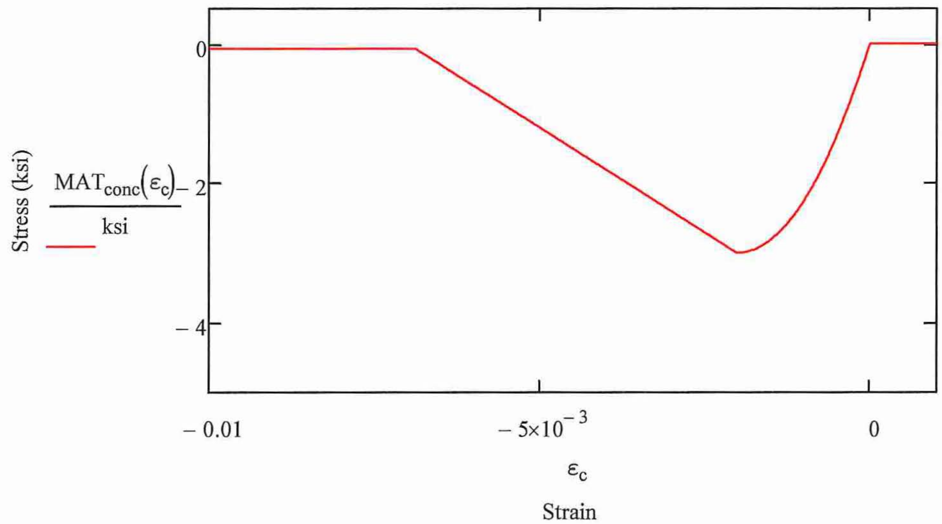
$$Z := \frac{0.5}{\varepsilon_{50u} - \varepsilon_{co}} = -200$$

Residual compressive strength

$$f_{c.res} := f_c \cdot 0.025 = -75 \cdot psi$$

Constitutive model for concrete

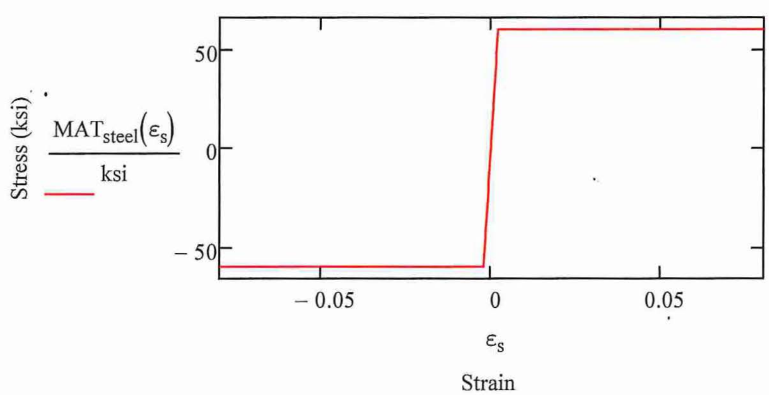
$$MAT_{conc}(\varepsilon) := \begin{cases} \min\left[f_{c.res}, f_c \cdot \left[1 - Z \cdot \left(\varepsilon - \varepsilon_{co}\right)\right]\right] & \text{if } \varepsilon < \varepsilon_{co} \\ f_c \cdot \left[\frac{2 \cdot \varepsilon}{\varepsilon_{co}} - \left(\frac{\varepsilon}{\varepsilon_{co}}\right)^2\right] & \text{if } \varepsilon_{co} \le \varepsilon < 0 \\ 0 & \text{if } 0 \le \varepsilon \end{cases}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{cases} f_y & \text{if } \varepsilon > \frac{f_y}{E_s} \\ -f_y & \text{if } \varepsilon < \frac{-f_y}{E_s} \\ \left(E_s \cdot \varepsilon\right) & \text{otherwise} \end{cases}$$



Concrete Fibers

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \frac{h}{Conc_{Num}} = 1.8 \cdot in$

Concrete fiber coordinates
$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\frac{h}{2} + \frac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain
$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress
$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force
$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -14.5 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 14.5 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 0.79 \cdot in^2$$

$$Steel_{As_2} := A_s = 0.79 \cdot in^2$$

Steel fiber strain
$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress
$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force
$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$$\text{Concrete}_{\sigma} := \text{Conc}_{\sigma}\left(\varepsilon_{o.conc}, 0, 0\right)$$

$$\text{Concrete}_{\sigma_1} = -0.135 \cdot \text{ksi}$$

Initial stress in steel

$$\text{Rebar}_{\sigma} := \text{Steel}_{\sigma}\left(\varepsilon_{o.steel}, 0, 0\right)$$

$$\text{Rebar}_{\sigma_1} = 38.873 \cdot \text{ksi}$$

Axial Equilibrium

$$\text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Conc}_{Num} \\ \quad \text{ans1} \leftarrow \text{ans1} + \text{Conc}_F\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_i \\ \text{ans2} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Steel}_{Num} \\ \quad \text{ans2} \leftarrow \text{ans2} + \text{Steel}_F\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_i \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Moment Equilibrium

$$\text{Moment}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi\right) := \left| \begin{array}{l} \text{ans1} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Conc}_{Num} \\ \quad \text{ans1} \leftarrow \text{ans1} + -1 \cdot \text{Conc}_F\left(\varepsilon_{o.conc}, \varepsilon, \varphi\right)_i \cdot \text{Conc}_{y_i} \\ \text{ans2} \leftarrow 0 \\ \text{for } i \in 1 .. \text{Steel}_{Num} \\ \quad \text{ans2} \leftarrow \text{ans2} + -1 \cdot \text{Steel}_F\left(\varepsilon_{o.steel}, \varepsilon, \varphi\right)_i \cdot \text{Steel}_{y_i} \\ \text{ans} \leftarrow \text{ans1} + \text{ans2} \end{array} \right.$$

Solution

Known parameters

Axial force $P := -32.3\text{kip}$

Iteration

Curvature $\phi := 0.0000022 \cdot \frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium

$$f(x) := \text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi\right) - P$$

$$\varepsilon_{cent} := \text{root}\left(f(x_o), x_o\right) = -2.724 \times 10^{-5}$$

Sectional forces

$$\text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = -32.3 \cdot \text{kip}$$

$$\text{Moment}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = 26.431 \cdot \text{kip} \cdot \text{ft}$$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_\varepsilon := \text{Steel}_\varepsilon\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 1.345 \times 10^{-3} \\ 1.281 \times 10^{-3} \end{pmatrix}$$

$$\text{Rebar}_\sigma := \text{Steel}_\sigma\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 39.008 \\ 37.158 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_F\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 30.816 \\ 29.354 \end{pmatrix} \cdot \text{kip}$$

$$\text{Concrete}_y := \text{Conc}_y$$

Concrete fiber stress and strain

$$\text{Concrete}_\varepsilon := \text{Conc}_\varepsilon\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$$

$$\text{Concrete}_\sigma := \text{Conc}_\sigma\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \ldots = -1.124 \times 10^{-4}$$
$$+ \text{Concrete}_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete

$$\sigma_{max.comp} := \text{MAT}_{conc}\left(\varepsilon_{max.comp}\right) = -0.328 \cdot \text{ksi}$$

H9. TABLES

There are no tables.

H10. FIGURES

There are no figures.

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Engineering of Structures and Building Enclosures

CLIENT:	NextEra Energy Seabrook	PROJECT NO:	170444
SUBJECT:	Evaluation of maximum stress in rebars of Seabrook structures	DATE:	Dec 2017
		BY:	RWKeene
		VERIFIER:	ATSarawit

APPENDIX I

TENSILE STRESS IN REBARS OF WEST PIPE CHASE STRUCTURE

I1. REVISION HISTORY

Revision 0: Initial document.

I2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute the maximum tensile stress that can form in the rebars of the West Pipe Chase (WPC) structure.

I3. RESULTS AND CONCLUSIONS

Table I1 summarizes the tensile stress in rebars of the WPC structure calculated at critical locations. The Maximum tensile stress is 44 ksi at the base of the WPC north wall subjected to the second in situ load combination (LC2).

I4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (Calc. 170443-CA-04 Rev. A).

I5. ASSUMPTIONS

I5.1 Justified assumptions

There are no justified assumptions.

I5.2 Unverified assumptions

There are no unverified assumptions.

I6. METHODOLOGY

The critical demands that control rebar tension in the WPC structure are horizontal moment and horizontal tension near the base of the WPC north wall. Finite element analyses were conducted to calculate the axial force and bending moment at these locations due to ASR load [I1].

To calculate the stress in rebars subjected to a combination of axial force and bending moment, sectional analysis based on fiber section method, as explained in calculation main body, is used. The calculation is conducted per 1 foot width of the walls, and each section is discretized into 20 fibers. An example calculation that evaluates the stress in the horizontal rebars at the base of the WPC north wall is presented in Section I8. The ASR expansion of the WPC north wall is included in the analysis to find the initial stress state due to internal ASR alone.

I7. REFERENCES

[I1] Simpson Gumpertz & Heger Inc., *Evaluation of the Main Steam and Feedwater West Pipe Chase and Personnel Hatch Structures,* 170443-CA-04 Rev. A, Waltham, MA, Nov 2017.

[I2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[I3] United Engineers & Constructors Inc., *Analysis and Design of MS&FW Pipe Chase - West*, EM-20, Rev. 7, February 1986

I8. COMPUTATION

I8.1. Strain in Steel and Concrete due to Internal ASR expansion

Input Data

ASR expansion

Measured crack index	$\varepsilon_{CI} := 0.24 \frac{mm}{m}$	
Threshold factor	$F_{thr} := 1.0$	

Material properties

Compressive strength of concrete	$f_c := -3ksi$	Ref. [I1]
Young's modulus of concrete	$E_c := 3120ksi$	
Yield strength of steel	$f_y := 60ksi$	
Young's modulus of steel	$E_s := 29000ksi$	

Geometry

Width of fibers	$b := 12in$	Ref. [I2]
Total thickness or height	$h := 24in$	
Area of concrete	$A_c := b \cdot h = 288 \cdot in^2$	
Area of tensile reinforcement (#11@12 in.)	$A_s := 1.56in^2$	
Number of reinforcement in row, e.g. equal to 2 for tensile and compressive	$Steel_{Num} := 2$	
Depth to reinforcement	$d := 20.3in$	

Finding the strain in steel and concrete by satisfying compatibility and equilibrium

	Initial Guess
Initial mechanical strain in concrete	$\varepsilon_{o.conc} := 0$
Initial strain in steel	$\varepsilon_{o.steel} := 0$
	Given
Compatibility equation	$F_{thr} \cdot \varepsilon_{CI} = \varepsilon_{o.steel} - \varepsilon_{o.conc}$
Equilibrium equation	$(E_c \cdot A_c) \cdot \varepsilon_{o.conc} + (E_s \cdot A_s \cdot Steel_{Num}) \cdot \varepsilon_{o.steel} = 0$
	$ans := Find(\varepsilon_{o.conc}, \varepsilon_{o.steel})$

Initial strain in concrete and steel

$$\varepsilon_{o.conc} := ans_1 = -2.196 \times 10^{-5}$$

$$\varepsilon_{o.steel} := ans_2 = 2.18 \times 10^{-4}$$

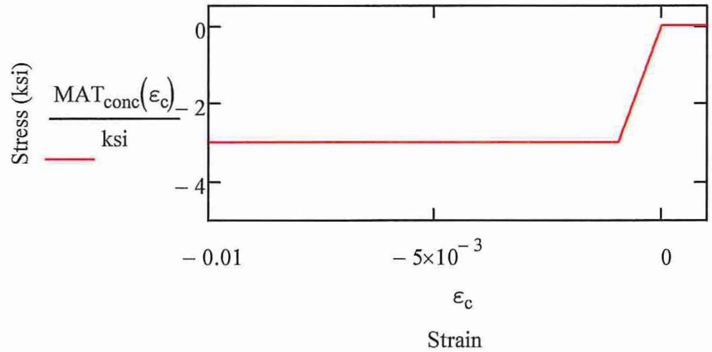
I8.2. Sectional Analysis

Input Data

Concrete Material Model

Constitutive model for concrete

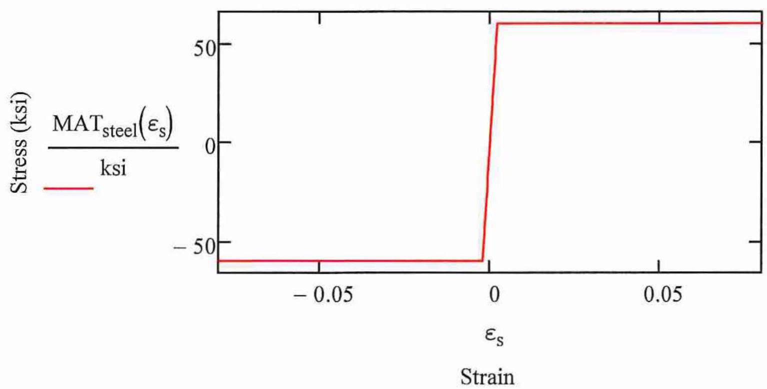
$$MAT_{conc}(\varepsilon) := \begin{vmatrix} 0 \ \text{ if } \varepsilon > 0 \\ f_c \ \text{ if } \varepsilon < \dfrac{f_c}{E_c} \\ (E_c \cdot \varepsilon) \ \text{ otherwise} \end{vmatrix}$$



Steel Material Model

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{vmatrix} f_y \ \text{ if } \varepsilon > \dfrac{f_y}{E_s} \\ -f_y \ \text{ if } \varepsilon < \dfrac{-f_y}{E_s} \\ (E_s \cdot \varepsilon) \ \text{ otherwise} \end{vmatrix}$$



Concrete Fibers

Number of fibers

$$Conc_{Num} := 20$$

Height of fibers

$$Conc_H := \frac{h}{Conc_{Num}} = 1.2 \cdot in$$

Concrete fiber coordinates

$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\frac{h}{2} + \frac{Conc_H}{2} + (i-1) \cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain

$$Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.conc} + \varepsilon - \varphi \cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress

$$Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_\varepsilon(\varepsilon_{o.conc}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force

$$Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_\sigma(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -8.3 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 8.3 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 1.56 \cdot in^2$$

$$Steel_{As_2} := A_s = 1.56 \cdot in^2$$

Steel fiber strain

$$Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_{o.steel} + \varepsilon - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress

$$Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_\varepsilon(\varepsilon_{o.steel}, \varepsilon, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force

$$Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_\sigma(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Initial Stress State

Initial stress in concrete

$Concrete_{\sigma} := Conc_{\sigma}(\varepsilon_{o.conc}, 0, 0)$

$Concrete_{\sigma_1} = -0.069 \cdot ksi$

Initial stress in steel

$Rebar_{\sigma} := Steel_{\sigma}(\varepsilon_{o.steel}, 0, 0)$

$Rebar_{\sigma_1} = 6.323 \cdot ksi$

Axial Equilibrium

$$Force(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \\ ans \leftarrow ans1 + ans2 \end{array} \right.$$

Moment Equilibrium

$$Moment(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon, \varphi) := \left| \begin{array}{l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + -1 \cdot Conc_F(\varepsilon_{o.conc}, \varepsilon, \varphi)_i \cdot Conc_{y_i} \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + -1 \cdot Steel_F(\varepsilon_{o.steel}, \varepsilon, \varphi)_i \cdot Steel_{y_i} \\ ans \leftarrow ans1 + ans2 \end{array} \right.$$

Solution

Known parameters

Axial force $P := 19.1\text{kip}$

Iteration

Curvature $\phi := 0.0000024 \cdot \frac{1}{\text{in}}$ Requires iteration

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.0$

Axial force equilibrium $f(x) := \text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, x, \phi\right) - P$

$$\varepsilon_{cent} := \text{root}\left(f\left(x_o\right), x_o\right) = 3.004 \times 10^{-5}$$

Sectional forces

$$\text{Force}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = 19.1 \cdot \text{kip}$$

$$\text{Moment}\left(\varepsilon_{o.conc}, \varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = 3.784 \cdot \text{kip} \cdot \text{ft}$$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$\text{Rebar}_{\varepsilon} := \text{Steel}_{\varepsilon}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 2.68 \times 10^{-4} \\ 2.282 \times 10^{-4} \end{pmatrix}$$

$$\text{Rebar}_{\sigma} := \text{Steel}_{\sigma}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 7.772 \\ 6.617 \end{pmatrix} \cdot \text{ksi}$$

$$\text{Steel}_{F}\left(\varepsilon_{o.steel}, \varepsilon_{cent}, \phi\right) = \begin{pmatrix} 12.124 \\ 10.322 \end{pmatrix} \cdot \text{kip}$$

$$\text{Concrete}_y := \text{Conc}_y$$

Concrete fiber stress and strain

$$\text{Concrete}_{\varepsilon} := \text{Conc}_{\varepsilon}\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$$

$$\text{Concrete}_{\sigma} := \text{Conc}_{\sigma}\left(\varepsilon_{o.conc}, \varepsilon_{cent}, \phi\right)$$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{\text{Concrete}_{\varepsilon_{ConcNum}} - \text{Concrete}_{\varepsilon_{ConcNum-1}}}{\text{Conc}_{y_{ConcNum}} - \text{Conc}_{y_{ConcNum-1}}} \cdot \left(\frac{h}{2} - \text{Conc}_{y_{ConcNum-1}}\right) \ldots = -2.072 \times 10^{-5}$$
$$+ \text{Concrete}_{\varepsilon_{ConcNum-1}}$$

Maximum compressive stress in concrete $\sigma_{max.comp} := \text{MAT}_{conc}\left(\varepsilon_{max.comp}\right) = -0.065 \cdot \text{ksi}$

I9. TABLES

Table I1: Stress in rebars at critical locations of WPC structure subjected to LC1

Component	Item	Total demands for sustained load (In Situ condition, LC1)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
WPC North Wall	Out-of-plane moment (kip-ft/ft)	3.8	Base of wall, horizontal direction	7.8	6.6	-0.07
	Axial force (kip/ft)	19.1				

Example in Section I8

Table I2: Stress in rebars at critical locations of WPC structure subjected to LC2

Component	Item	Total demands for sustained loads plus OBE amplified with threshold factor (In Situ condition, LC2)		Total stress in steel (ksi)		Maximum compressive stress in concrete (ksi)
		Demand	Location	Rebar 1	Rebar 2	
WPC North Wall	Out-of-plane moment (kip-ft/ft)	78.8	Base of wall, horizontal direction	44.4	8.0	-1.36
	Axial force (kip/ft)	34.4				

I10. FIGURES

There are no figures.

APPENDIX J
COMPUTER RUN IDENTIFICATION LOG

SIMPSON GUMPERTZ & HEGER
Engineering of Structures
and Building Enclosures

Client: NextEra Energy Seabrook **Page** 1 **of** 4

Project: Evaluation of maximum stress in rebars of Seabrook structures

Project No.: 170444 **Subcontract No.:** N/A **Calculation No.:** RAI-D8 Attachment 2

Run No.	Title	Program/Ver.[A]	Hardware	Date	Files	
1	CRMAI subjected to unfactored load (sustained loading) including ASR load	ANSYS 15.0 Structural	Cluster3g[B]	10/12/2017	Note C	
2	CRMAI subjected to unfactored load (sustained loading) plus OBE including ASR load that has been amplified by threshold factor .	ANSYS 15.0 Structural	Cluster3g[B]	10/12/2017	Note C	
3	CEB Standard Case subjected to unfactored loads (sustained loading) including ASR and OBE. For OBE load case, ASR loads are amplified by threshold factor.	ANSYS 15.0 Structural	Cluster3g[B]	11/22/2017	Note C	
4	CEB Standard-Plus Case subjected to unfactored loads (sustained loading) including ASR and OBE. For OBE load case, ASR loads are amplified by threshold factor.	ANSYS 15.0 Structural	Cluster3g[B]	11/22/2017	Note C	
5	RHR subjected to unfactored sustained loads (i.e., non ASR loads), unfactored ASR loads, and unfactored seismic loads considering unit acceleration (i.e., 1 g).	ANSYS 15.0 Structural	Cluster3g[B]	10/12/2017	Note C	
6	CSTE subjected to ASR load	ANSYS 15.0 Structural	Cluster3g[B]	11/11/2016	Note C	

SIMPSON GUMPERTZ & HEGER

Engineering of Structures and Building Enclosures

PROJECT NO: 170444

DATE: Dec. 2017

CLIENT: NextEra Energy Seabrook

BY: MR. M. Gargari

SUBJECT: Evaluation of maximum stress in rebars of Seabrook structures

VERIFIER: A.T. Sarawit

Run No.	Title	Program/Ver.[A]	Hardware	Date	Files	
7	WPC/PH subjected to ASR load	ANSYS 15.0 Structural	Cluster3g[B]	11/09/2017	Note C	

Notes:

A ANSYS 15.0 Structural is QA verified

B Cluster3g information is provided below:
Model: Compute Blade E55A2
Serial Number: 4600E70 T201000293
Manufacturer: American Megatrends Inc.
Operating System: Microsoft Windows NT Server 6.2 (x64)

C Input and output files for ANSYS computer runs are listed in Table J1.

Table J1. Input and output files for ANSYS computer runs

Run No.	Input Files[A]	Output Files[A]
1	CRMAI_SUS.db[B]	CRMAI_SUS.rst
2	CRMAI_SUS_OBE.db[B]	CRMAI_SUS_OBE.rst
3	SR_Rebar_Stress_A10_r0.db[B]	SR_Rebar_Stress_A10_r0.lxx
4	SR_Rebar_Stress_B7_r0.db[B]	SR_Rebar_Stress_B7_r0.lxx
5	Non ASR Loads • RHR_ILC_02.db • RHR_ILC_03.db • RHR_ILC_05.db • RHR_ILC_16.db ASR Loads • RHR_ILC_09.db • RHR_ILC_10.db Seismic 1g • RHR_ILC_06.db • RHR_ILC_07.db • RHR_ILC_08.db • RHR_ILC_13.db • RHR_ILC_14.db	Non ASR Loads • RHR_ILC_02.rst • RHR_ILC_03.rst • RHR_ILC_05.rst • RHR_ILC_16.rst ASR Loads • RHR_ILC_09.rst • RHR_ILC_10.rst Seismic 1g • RHR_ILC_06.rst • RHR_ILC_07.rst • RHR_ILC_08.rst • RHR_ILC_13 rst • RHR_ILC_14.rst
6	CST_024.db[C]	CST_024.rst
7	WPC.db[C]	WPC.rst

Notes:

A Input and output files are provided on RAI-Attachment-CD. File type descriptions are as follows.
*.db = ANSYS database file containing the model (nodes, elements, properties, boundary conditions, loads, etc.).
*.rst = ANSYS result file containing forces, moments, reactions, displacements, etc.
*.lxx = ANSYS load case file containing forces, moments, reactions, displacements, and other structural response output for load cases and load combinations.

B Each structure has been analyzed for two load combination as follows:
- D + L + E + To + Sa (In-situ condition, LC1)
- $D + L + E + T_o + E_o + He + F_{THR}.S_a$ (In-situ condition plus seismic load, LC2)

C Each structure is analyzed for ASR load only. The original design demands are extracted from original design calculation.

D The description of the input and output files for Run No. 5 is following:
RHR_ILC_02: Self-Weight:
RHR_ILC_03: Hydrostatic Pressure Outside
RHR_ILC_05: Live Load
RHR_ILC_06: Seismic North-South with 1g acceleration
RHR_ILC_07: Seismic East-West with 1g acceleration
RHR_ILC_08: Seismic Vertical with 1g acceleration
RHR_ILC_09: In structure ASR

RHR_ILC_10: Concrete fill
RHR_ILC_13: Seismic South-North with 1g acceleration
RHR_ILC_14: Seismic West-East with 1g acceleration
RHR_ILC_16: Backfill Soil Static Pressure

Enclosure 3 to SBK-L-17204

NextEra Energy Seabrook Updated Final Safety Analyses Report Markup of Sections 1.8 and 3.8 – Design of Structures, Components, Equipment and Systems – Design of Category I Structures

For the safety-related equipment located inside the containment and required after a LOCA, the sources used in establishing the integrated dose are consistent with Regulatory Guide 1.89, Rev. 0. Both instantaneous gamma and beta doses have been considered in establishing the integrated doses.

The seismic qualification of electrical, instrumentation and control equipment meets the requirements of IEEE 344-1975.

Conformance of NSSS Class 1E equipment with IEEE Standard 323-1974 (including IEEE Standard 323A-1975 position statement of July 23, 1975) and Regulatory Guide 1.89 is being demonstrated by an appropriate combination of any or all of the following: type testing, operating experience, qualification by analysis and on-going qualification programs. This commitment is being satisfied by implementation of Reference 9.

Regulatory Guide 1.90

<u>(Rev. 1, 8/77)</u> <u>Inservice Inspection of Prestressed Concrete Containment Structures with Grouted Tendons</u>

This guide is not applicable to Seabrook Station since grouted tendons are not employed.

Regulatory Guide 1.91

<u>(Rev. 1, 2/78)</u> <u>Evaluation of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants</u>

The guidance provided by Regulatory Guide 1.91, Rev. 1, has been followed. For further discussion, refer to Section 2.2.

Regulatory Guide 1.92

<u>(Rev. 1, 2/76)</u> <u>Modes and Spatial Components in Seismic Response Analyses</u>

The procedure for combining modal responses for NSSS equipment is presented in Subsection 3.7(N).3.2. This procedure is considered as an alternate acceptable solution that provides a basis for findings requisite to issuance or continuance of a permit or license by the NRC.

The procedure used for BOP equipment design fully conforms with the guidance of Regulatory Guide 1.92, Rev. 1, relative to seismic system analysis. The requirements for the combination of modes and spatial components in seismic subsystem analyses also complies with this guide, except that closely spaced modes for seismic analyses of components by the normal mode approach are combined in accordance with the two methods indicated in Subsection 3.7(B).3.6.

Regulatory Guide 1.92 presents three other means of combining closely spaced modes. Justification for nonconformance is that the methods prescribed in the guide are not here applicable since the construction permit application docket date is before the date of issue of the guide. In addition, the method used is deemed more conservative. For further discussion, refer to Section 3.7(B) and Subsection 3.7(B).3.7.

~~(Rev. 3, 10/2012) Modes and Spatial Components in Seismic Response Analyses~~

~~A procedure for combining the three spatial components of an earthquake for seismic response analysis of nuclear power plant structures, systems, and components (SSCs) that are important to safety is presented in Subsection C.2.1. The Response Spectrum Method that uses the 100-40-40 percent combination rule, as described in Regulatory Position C.2.1 of this guide, is acceptable as an alternative to the SRSS method.~~

~~The 100-40-40 percent rule is used as an alternative to the SRSS method for combining three directional seismic loading in the analysis of seismic, Category I structures that are deformed by the effects of ASR. In general, the 100-40-40 combination method produces higher estimates of maximum response than the SRSS combination method by as much as 16 percent, while the maximum under-prediction is 1 percent.~~

~~Refer to Section 3.7(B).2.1 for further discussion of this subject.~~

Regulatory Guide 1.93

<u>(Rev. 0, 12/74)</u> <u>Availability of Electric Power Sources</u>

The Technical Specification (T/S) ac and dc power sources allowable out-of-service times (action statements) are based on RG 1.93. Where differences exist between the T/S and RG 1.93, the T/S are the governing document.

RG 1.93 does not allow out-of-service times to be used for preventative maintenance that incapacitates a power source. These activities are to be scheduled for refueling or shutdown periods. This is interpreted to also apply to surveillance activities. Preventative maintenance and surveillance activities are performed on-line when permitted by the T/S and with appropriate consideration of the effects on safety, reliability, and availability.

Regulatory Guide 1.95

<u>(Rev. 1, 2/77)</u> <u>Protection of Nuclear Power Plant Control Room Operators Against Accidental Chlorine Release</u>

The relevant portions of Regulatory Guide 1.95 are complied with based on the findings that the plant design does not include the storage of chlorine within 100 meters of the control room, excluding small laboratory quantities, nor is there chlorine stored in excess of the maximum allowable chlorine inventory, as given as a function of distance in Regulatory Guide 1.95 for Type I control rooms (refer to Subsection 2.2.3.1).

3.7(B).2 Seismic System Analysis

This subsection contains a discussion of the seismic analyses performed for seismic Category I structures and systems. Included in the discussion are the methods of seismic analysis used, the criteria used for mathematically modelling the structures and systems, the assumptions made in the analyses, and the effects considered.

3.7(B).2.1 Seismic Analysis Methods

The seismic response of Category I structures, systems and components has been determined from suitable elastic dynamic analyses. The results of these analyses are used for the design of seismic Category I structures, systems and components, and are input for subsequent dynamic analyses.

Two methods of seismic system analysis were used for seismic Category I structures: (1) the modal analysis response-spectrum method and (2) the mode-superposition time-history method. The time-history method was used to determine the dynamic response necessary to obtain amplified response spectra for component design. The input forcing functions (the time history of ground motion) are shown graphically in Figure 3.7(B)-1, Figure 3.7(B)-2 and Figure 3.7(B)-3. The time history shown on Figure 3.7(B)-1 is used in both horizontal directions. The peak acceleration is 0.25g for the SSE and 0.125g for the OBE. Design response spectra for the response-spectrum method are shown in Section 2.5.

The mathematical models used for the seismic Category I structures are typically lumped masses connected by linear elastic springs. Each structure, then, is described by a finite number of degrees-of-freedom chosen to represent the principal overall behavior of the system. The modelling is described in Subsection 3.7(B).2.3 in more detail. The number of masses or degrees-of-freedom included in the analysis is determined by requiring the total degrees-of-freedom to exceed twice the number of significant modes with frequencies less than 33 Hz. Up to four degrees-of-freedom were considered for each mass point, three translation and one torsion. The three orthogonal directions were run separately, and results were combined by the grouping method in accordance with Regulatory Guide 1.92~~. The orthogonal spatial components of seismic loads for response spectrum analyses of structures deformed by the effects of ASR are combined using the 100-40-40 procedure in Regulatory Guide 1.92 Revision 3.~~

All significant modes with frequencies up to 50 Hz were used in analyses for both local and overall effects.

The effects due to inertial characteristics of fluid contained within a structural component were considered in the analysis by techniques described in Reference 1. No soil-structure interaction effects were involved because of the rock siting.

TABLE 3.8-17 COMPUTER PROGRAMS USED IN THE ANALYSIS AND DESIGN OF OTHER SEISMIC CATEGORY I STRUCTURES

Computer Program	Structures On Which Used
1. MRI/STARDYNE (Static Analysis)	Control & Diesel Generator Building
	Fuel Storage Building
	Main Steam and Feedwater Pipe Chase (East)
	Main Steam and Feedwater Pipe Chase (West)
	Pre-Action Valve Area
	Primary Auxiliary Building Including Residual Heat Removal Equipment Vault
	Service Water Cooling Towers Including Switchgear Room
	Service Water Pumphouse
2. MARC-CDC (Static Analysis)	Containment Enclosure Building
3. LESCAL (Design of Reinforcing Steel)	Containment Enclosure Building
	Main Steam and Feedwater Pipe Chase (East)
4. GENSAP (Static Analysis)	Containment Enclosure Ventilation Area
	Emergency Feedwater Pump Building Including Electrical Cable Tunnels and Penetration Areas
	Piping Tunnels
5. MULTISPAN (Static Analysis)	Service Water Cooling Towers
6. ANSYS (ASR Deformation Analysis)	All Seismic Category I Structures ~~Containment Enclosure Building~~

g. Description of the Computer Programs Utilized in the Design and Analyses

The computer programs used in the containment design and analysis are briefly described in this subsection. ~~A summary of the comparisons of results used to validate them is given in Appendix 3F.~~ The programs are either of two types: a recognized program in the public domain with sufficient history of use and documentation to justify its applicability and validity without further demonstration, or a program which gives solutions to a series of test problems that have been demonstrated to be substantially identical to those obtained from classical solutions and/or analytical results published in technical literature. Utility programs used to replace hand calculations are not discussed. These programs were validated by comparison to sample hand calculations whenever used in the analysis.

1. STARDYNE, Static and Dynamic Structural Analysis System, by Mechanics Research, Inc., 9841 Airport Boulevard, Los Angeles, CA., 90045. Documentation is available from Control Data Corporation (Publication No. 76079900). The STARDYNE system is designed to analyze linear elastic structural models for a wide range of static and dynamic problems.

2. MARC-CDC, Nonlinear Finite Element Analysis Program, by Dr. Pedro Marcel and Associates of the MARC Analysis Corporation, 260 Sheridan Ave., Palo Alto, CA., 94306. Documentation is available from Control Data Corporation. MARC-CDC provides elastic, elastic-plastic, creep, large displacement, buckling and heat transfer analysis capabilities. It also performs dynamic analysis by the modal or direct integration procedures.

3. WILSON 1, (SAG 001) Finite Element Analysis of Axisymmetric Solids Subjected to Axisymmetric Loads, by E. L. Wilson of the University of California, Berkeley, July 1967-Revised, November 1969. Documentation is available from the Earthquake Engineering Research Center of the University of California, Berkeley. The Wilson 1 computer program is based on the finite element direct stiffness method, and is applied to the determination of stresses and displacements in axisymmetric structures (solids and/or shells of revolution) subjected to axisymmetric mechanical loads or temperature gradients. The theoretical basis of the program is the work of E. L. Wilson, References 1 and 2.

4. WILSON 2, (SAG 010), Dynamic Stress Analysis of Axisymmetric Structures under Arbitrary Loading, by S. Ghosh and E. L. Wilson of the University of California, Berkeley, September, 1969-Revised September 1975. Documentation is available from the Earthquake Engineering Research Center of the University of California, Berkeley, Report, No. EERC 69-10. The Wilson 2 program is based on the finite element stiffness method and is applied to complex axisymmetric structures subjected to any arbitrary static or dynamic loading or base excitation. The three-dimensional axisymmetric continuum is represented as an axisymmetric thin shell, a solid of revolution, or as a combination of both.

5. LESCAL calculates the stresses and strains in rebars and/or concrete in accordance with the criteria set forth in Subarticle CC-3511 of Division 2. The section may be reinforced with horizontal, vertical and diagonal rebars. The applied loads are axial forces and moments in the vertical and horizontal faces and in-plane shear. When in-plane shear forces are included, a solution is obtained by solving Duchon's equations, Reference 3.

6. SAG 054, Amplified Floor Response Envelope. This program generates an envelope for amplified response spectra, spreading the peaks by a user-specified amount.

7. SAG 058, Response Spectra. This program calculates the response spectra of a single degree-of-freedom damped oscillator due to a transient base motion. The input base motion may be an arbitrary forcing function. The output consists of the maximum relative displacement, the maximum relative velocity and the maximum absolute acceleration for the various selected frequencies and the times when these values occur.

8. TAPAS, (SAG 008), "Transient Temperature Analysis of Plane and Axisymmetric Solids," Reference 4, was developed to determine the temperature distribution through a solid body as a function of time when subjected to temperature variation or heat flux inputs. A finite element technique coupled with a step-by-step time integration procedure is used. Both steady-state and transient heat flow can be treated.

9. SAG 017, Fourier Coefficient Expansion Program, was developed to be used in conjunction with the Wilson 2 program to compute Fourier series representation of general nonaxisymmetric load functions.

10. SAG 024, MMIC, calculates weight, weight moments of inertia and plan location of the center of weight of a segment of a structure given the dimensions, density and location of each structural component and the magnitude and location of all concentrated loads.

11. SAG 025, SECTION, calculates beam section properties of structures for use in lumped mass stick models for dynamic analysis.

12. ANSYS, Finite Element Analysis Program, by ANSYS, Inc., Canonsburg, PA, USA. ANSYS is a general purpose finite element commercial software. It provides broad range of analysis capabilities including static and dynamic analyses. This program is used for ASR deformation analysis.

(Section 3.8.3.3.b)

b. Load Combinations

Various load combinations are considered in design to determine the greatest strength requirements of the structure. Where varying loads occur, the combinations producing the most critical loading are used. Basic combinations in the design of the containment internal structures are given in Table 3.8-14. These load combinations are in agreement with Subsections II.3 and II.5 of the Standard Review Plan for Subsection 3.8.3 of the UFSAR. The factors which are to be applied to allowable stresses have been transposed and applied as load factors instead, resulting in acceptance criteria as indicated in the table. Two categories of loading conditions and criteria are used in the design of the containment internal structures as described below.

1. Normal Load Conditions

Normal load conditions are those encountered during testing and normal operation. They include dead load, live load and anticipated transients or test conditions during normal and emergency startup and shutdown of the Nuclear Steam Supply, Safety and Auxiliary Systems. Normal loading also includes the effect of an Operating Basis Earthquake. Normal load conditions are referred to in Division 2 as service load conditions.

Under each of these unfactored loading conditions, the structure is designed so that the behavior of structure is in the small deformation elastic range~~stresses will be within the elastic limits~~. Design assumptions are presented in Subsection 3.8.3.4 and stress limitations are presented in Subsection 3.8.3.5.

2. Unusual Load Conditions

Unusual load conditions are those conditions resulting from combinations of the LOCA, SSE and OBE, high-energy pipe failures, and live and dead loads. They are referred to in Division 2 as factored load conditions.

For each of the unusual loading combinations, the internal structures are designed to remain below their ultimate capacity so that the behavior of structural components is in the small deformation elastic range. Design assumptions are presented in Subsection 3.8.3.4 and stress limitations are presented in Subsection 3.8.3.5.

3.8.3.5 Structural Acceptance Criteria

The bases for the development of the following stress-strain criteria are the ACI 318-71 and AISC codes.

a. Normal Load Conditions

Internal structures are proportioned to maintain elastic behavior ~~remain within the elastic limits~~ under all normal loading conditions described in Subsection 3.8.3.3.

Reinforced Concrete - designed in accordance with ACI 318-71 Strength Method, which insures flexural ductility by control of reinforcing steel percentages and stresses

Structural and Miscellaneous Steel - designed in accordance with AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, Part I.

b. Unusual Load Conditions

Internal structures are designed to maintain elastic behavior under all unusual load conditions shown in Subsection 3.8.3.3. The upper bound of elastic behavior is taken as the yield strength capacity of the load carrying components. The yield strength of structural and reinforcing steel is taken as the minimum guaranteed yield stress as given in the appropriate ASTM Specification.

Reinforced Concrete - designed in accordance with ACI 318-71 Building Code. Member yield strength is considered to be the strength capacity calculated by the ACI Code.

Structural and Miscellaneous Steel - designed in accordance with AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, Part 1.

Overall stability of steel structures designed for unusual loading is verified using the AISC Specification, Part 2, and the load factors in Table 3.8-14, with the exception of the reactor vessel lateral support, which is designed in accordance with ASME B&PV Code, Division 1, Subsection NF.

(Section 3.8.4.3.b)

b. Load Combinations

Various load combinations were considered in design to determine the strength requirements of the structure. Where varying loads occur, the combinations producing the most critical loading were used. The basic combinations considered in the design of each seismic Category I structure are given in Table 3.8-16.

Two categories of loading conditions and criteria were used in the design of the seismic Category I structures other than the containment, as described below:

1. Normal Load Conditions

   Normal load conditions are those encountered during testing and normal operation and are referred to in the standard review plan as service load conditions. They include dead load, live load and anticipated transients, loads occurring during normal startup and shutdown, and loads occurring during emergency shutdown of the nuclear steam supply, safety and auxiliary systems. Normal loading also includes the effect of an operating basis earthquake and normal wind load. Under each of these loading combinations the structures were designed such that deformations will be small and the structure will respond elastically~~so that stresses are within the elastic limits~~. Design and analysis procedures are presented in Subsection 3.8.4.4 and stress limitations are presented in Subsection 3.8.4.5.

2. Unusual Load Conditions

   Unusual load conditions are those resulting from combinations of accident, wind, tornado, earthquake, live and dead loads and are referred to in the standard review plan as factored load conditions.

   For these loading combinations, the structures were designed to remain below their ultimate yield capacity such that deformations will be small and structural components will respond elastically. Design and analysis procedures are presented in Subsection 3.8.4.4 and stress limitations are presented in Subsection 3.8.4.5.

3.8.4.5 Structural Acceptance Criteria

The basis for the acceptance criteria is the ACI 318 and AISC Codes. However, under the action of seismic or wind loadings, in accordance with the standard review plan (Section II.5), the 33 percent increase in allowable stresses was not permitted.

a. Normal Load Conditions

Structures were proportioned to maintain elastic behavior ~~remain within the elastic limits~~ under all normal loading conditions described in Subsection 3.8.4.3. Reinforced concrete structures were designed in accordance with ACI-318 Strength Method, which insures flexural ductility by limiting reinforcing steel percentages and stresses.

Structural and miscellaneous steels were designed in accordance with AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, Part 1.

b. Unusual Load Conditions

Structures were proportioned to maintain elastic behavior under all unusual load conditions shown in Subsection 3.8.4.3. The upper bound of elastic behavior was taken as the yield strength capacity of the load carrying components. The yield strength of structural and reinforcing steel was taken as the minimum guaranteed yield stress as given in the appropriate ASTM Specifications. Reinforced concrete structures were designed in accordance with ACI-318 Building Code. Member yield strength was considered to be the strength capacity calculated by the ACI Code.

Structural and miscellaneous steels were designed in accordance with Part 1 of AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.

c. Deformations

Since each of the structures was designed to be in the small deformation, elastic range, no gross deformations will occur that will cause significant contact with other structures or pieces of equipment. All deformations, however, were evaluated considering the relationship of the subject component to both adjacent and supporting structures and equipment.

Enclosure 4 to SBK-L-17204

Methodology for the Analysis of Seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction for Seabrook Station

METHODOLOGY FOR THE ANALYSIS OF SEISMIC CATEGORY I STRUCTURES WITH CONCRETE AFFECTED BY ALKALI-SILICA REACTION

December 2017

TABLE OF CONTENTS

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

SYMBOLS AND ABBREVIATIONS

ACI	American Concrete Institute
AR	Action Request
ASME	American Society of Mechanical Engineers
ASR	Alkali-Silica Reaction
ASTM	American Society for Testing and Materials
CCI	Combined Cracking index
CI	Cracking Index
f_c'	Compressive strength of concrete
FSEL	Ferguson Structural Engineering Laboratory
LAR	License Amendment Request
LOCA	Loss-Of-Coolant Accident
MPR	MPR Associates, Inc.
NextEra	NextEra Energy Seabrook
NRC	Nuclear Regulatory Commission
OBE	Operating Basis Earthquake
SGH	Simpson Gumpertz & Heger Inc.
SMP	Structural Monitoring Program
SRSS	Square Root of the Sum of the Squares
SSE	Safe-Shutdown Earthquake
UFSAR	Updated Final Safety Analysis Report
WO	Work Order

REVISION HISTORY

Revision 0: Initial document.

1. INTRODUCTION

1.1 Purpose

The purpose of this methodology document is to provide a method for analyzing and evaluating Seismic Category I structures with concrete affected by ASR. This analysis methodology is in accordance with the approach outlined in License Amendment Request 16-03 (LAR) [1], which NextEra Energy Seabrook (NextEra) submitted to the NRC in 2016. Details of these analysis and evaluation procedures permit all Seismic Category I structures to be evaluated using the same methodology, and provide clear guidance to engineers performing the structural evaluation with sufficient procedural details to be repeatable by other engineers.

NextEra initially identified pattern cracking typical of alkali-silica reaction (ASR) in the B Electrical Tunnel in 2009, and, subsequently, in several other Seismic Category I structures at Seabrook Station. NextEra informed the Nuclear Regulatory Commission (NRC) of this discovery and then performed a root cause investigation into the presence of ASR at Seabrook Station. The root cause investigation concluded that the original concrete mix designs used a coarse aggregate that was susceptible to ASR. An interim structural assessment was completed in 2012 [2]. The evaluation concluded that the reinforced concrete structures at Seabrook Station remain suitable for continued service for an interim period given the extent of ASR identified at that time. The evaluation noted that additional testing was required to verify that some structures satisfy ACI 318-71 [3] code requirements for shear and reinforcement anchorage (development and lap length). NextEra has since completed MPR/FSEL large-scale test programs [4] and used the test results and literature to develop guidance for evaluating ASR-affected reinforced concrete structures at Seabrook Station [5].

Seismic Category I structures other than the Containment Structure were originally designed to meet the requirements of ACI 318-71. The Containment Structure is a reinforced concrete structure that was designed in accordance with the requirements of Section III of the American Society of Mechanical Engineers (ASME) Boiler & Pressure Vessel Code (1975 Edition) [6]. Neither ACI 318-71 nor the ASME code include provisions for the analysis and evaluation of structures affected by ASR. The analysis approach established in this methodology document properly accounts for the effects of ASR when performing the building evaluations. Following the methodology and meeting the acceptance criteria that are established herein will demonstrate that the structure meets the intent of the original design codes of record and achieves the structural safety reliability indices consistent with the original design. This methodology document also provides a list of deviations from the codes of record and justification that those deviations meet the intent of the original codes of record.

Development of ASR loads based on field measurements of in-plane ASR expansion within the structure and assessment of concrete backfill ASR expansion establishes a set of loads applicable to the time concurrent with the measurements. The potential for future ASR expansion is considered in the methodology by stipulating procedures for reevaluation of the structure for an increased ASR load to account for potential future ASR expansion. When the evaluation of a structure based on current ASR demands consistent with current inspection data indicates that remaining design margin exists, the evaluation to confirm the current design margin shall be performed with ASR loads amplified by a threshold factor such that the controlling demands on the structure are equal to (or slightly less than) the capacity of the structure.

The threshold factor represents the reserve design margin in the structure for accommodating increasing ASR future demands. Once the threshold factor is established, the structure is monitored, and quantitative measurements and qualitative observations are compared to the specified limit. If the quantitative measurements or qualitative observations approach the corresponding specified limits, then further structural evaluations in accordance with procedures specified in this methodology document are performed to reevaluate the structure in order to increase the threshold limit without decreasing code inherent safety factors, or structural modifications may be made to alleviate the concern for the approaching threshold limit.

1.2 Scope

The scope of this document is to define a methodology for consistent and repeatable evaluation of the concrete structures and foundations of Seismic Category I structures at Seabrook Station that are impacted by self-straining loads including ASR. This methodology for structural evaluation includes the use of dimensions, details, and notes on the structural design drawings. This document defines the material properties, loads and load combinations, analysis methods, acceptance criteria, threshold factor, and actions to follow when the structural monitoring program shows that the ASR growth is reaching the threshold limits. Because of flexibility and tolerances in structural steel construction, structural deformations due to ASR expansion are considered to be small enough to not impact the design or integrity of the steel structures supported on the reinforced concrete structures or foundations affected by ASR. Possible impacts of concrete deformations due to ASR on equipment are beyond the scope of this document.

1.3 Document Organization

This methodology document is organized into following sections:

Section 1: Introduction – Provides an overview of the analysis and evaluation methodology and defines the purpose and scope.

Section 2: Characteristics and Measurement of ASR – Provides background information on effects of ASR on reinforced concrete material properties and measurement methods.

Section 3: Loads and Load Combinations – Discusses the addition of self-straining loads (creep, shrinkage, and swelling) to the original design loads and the addition of ASR load to the design load combinations.

Section 4: Analysis Approach – Provides methodologies for Stage One, Two, and Three analyses which include field observations to support the analysis, calculation for non-ASR and ASR demands, analysis method, and correlation of analysis results to field observations.

Section 5: Acceptance Criteria – Defines the acceptance criteria for evaluation of Seismic Category I structures and identifies deviations from the original codes of record and the reason and justification for those deviations.

Section 6: ASR Threshold Limits and Monitoring – Discusses methodology to account for potential future ASR expansion and determine the ASR threshold limit, and methods to monitor the structures.

Section 7: Evaluation for Approaching Threshold Limits – Discusses the evaluation procedures and decision options to alleviate the concern for structures approaching ASR threshold limits.

Section 8: References – Provides a complete list of cited references.

1.4 Overview of Methodology

1.4.1 Analytical and Evaluation Methods

A three-stage analysis approach shall be used for analyzing and evaluating Seismic Category I structures, as identified in the Seabrook LAR 16-03. Each stage applies more sophisticated methods and uses additional field measurement data of ASR expansion to improve the rigor of the analysis. The analysis and evaluation of each structure may begin at any stage and, if necessary, shall progress to a higher analysis stage. The three analysis stages are described in Section 4.

Each structure shall be analyzed in accordance with the required stage of analysis. The total demands, including the ASR demands amplified by threshold limits, will be compared to the acceptance criteria described in Section 5. The threshold monitoring limits will be determined based on the structural evaluation calculations that are specific for each structure and shall be included in the Structural Monitoring Program (SMP), as discussed in Section 6. The evaluation for approaching the ASR threshold limits are discussed in Section 7.

1.4.2 Effects of ASR on Reinforced Concrete Material Properties

As reported in the technical literature [7] and observed in the MPR/FSEL test programs [4], one effect of ASR is that material properties of unreinforced concrete are reduced. Specifically, compressive strength exhibits a relatively shallow decrease as a function of ASR progression, while elastic modulus and tensile strength are much more sensitive. However, for reinforced concrete structures, this decrease in material properties may not result in a decrease in structural performance, due to the chemical prestressing effect that also results from ASR [7]. For the limit states of shear and flexure, results from the MPR/FSEL test programs suggest that the effects of confinement from the reinforcement more than compensate for degradation of material properties when ASR progression is within the range observed in the test specimens [5].

From the MPR studies referenced above, it is concluded that for analyses of reinforced concrete structures at Seabrook Station, the elastic modulus of concrete shall be computed using the minimum specified design compressive strength, and no reduction shall be taken for ASR-related damage. Material properties for analysis and evaluation of Seismic Category I reinforced concrete structures shall be consistent with those used in the original design calculations [8] and those defined in the project specifications and drawing notes.

2. CHARACTERISTICS AND MEASUREMENT OF ASR

Alkali-silica reaction (ASR) is a chemical reaction between the alkali content in cement paste and reactive silica minerals contained in some types of concrete aggregates. The reaction produces a gel that swells if moisture is present. Typical characteristics of ASR include some combination of expansion and displacement of elements, a characteristic pattern of interconnected and closely-spaced cracks (which can be influenced by the stress state of the concrete), surface discoloration of cracks, ASR gel exudations, and an environment conducive to the development of ASR. ASR can cause concrete cracking, structural deformations and stresses, changes in unreinforced concrete strength and/or stiffness, and external loads on other adjacent structures.

ASR can be identified in the field through qualitative visual observations of common symptoms as reported in industry guidelines [9] and [10]. These guidelines include a classification system for the Condition Survey, which establishes the correlation between ASR features and probability of ASR. The field evaluation of the "likelihood" of ASR can be classified as "No", "Possible", or "Likely". The specific visual features that are suggestive or indicative of ASR are summarized in Table 1 (taken from Table 2 of CSA International document A864-00 [10]).

Field Inspectors shall use the above-mentioned industry guidelines to identify the likelihood of ASR. Locations for which the criteria indicate that ASR is "Possible" or "Likely" shall be accounted for in the structural evaluation unless petrographic examinations of concrete samples extracted from that location show otherwise. Petrographic investigation can confirm ASR features, such as identification of aggregate types and reactive components, general characterization of microcracking, presence of reaction and/or the presence of an alteration rim around aggregate particles, and presence of gel or other deposits in voids.

Quantitative measurement of ASR in-plane expansion can be made by summation of crack widths or by measurement of change in distance between two points as explained in the following subsections.

2.1 Cracking Index (CI)

The cracking Index (CI) is a quantitative measurement of ASR in-plane progression obtained by crack width summation and normalization. It includes measurement and summation of crack widths along a set of perpendicular reference lines on the surface of a concrete element being investigated. The sum of crack widths is normalized by the length of the reference lines to determine the CI in-plane expansion. The CI measurement captures surface crack widths from all sources of cracking and thus can be influenced by cracks from phenomena other than from ASR. The Combined Cracking Index (CCI) is the weighted average of the CI in the two measured in-plane directions. A typical ASR-monitoring location produces two CI values (in-plane perpendicular directions) and one CCI value. CI and CCI values are typically reported in mm/m. The CCI provides a reasonable approximation of true engineering strain and is acceptable for monitoring in-plane strain [11] and [4]. Measurement of concrete expansion can be approximated by crack width summation because concrete has low capacity for expansion before cracking. While true engineering strain is represented by the sum of material elongation and crack widths, the crack width term generally dominates the overall expansion. The Seabrook SMP uses CI to establish baseline strain in ASR deformed concrete.

2.2 Pin-to-Pin In-Plane Expansion Measurements

In-plane expansion measurements are quantitative measurements of the distance between two points (pin-to-pin) installed on the surface of a concrete component using a removable strain gage. Pin-to-pin in-plane expansion is computed as the change in length-measurement values recorded at different times.

Pin-to-pin measurements determine changes in ASR expansion more precisely than CI measurements determine changes over the duration of the monitoring period, since they are performed using a calibrated mechanical device capable of measuring changes in length as small as 0.0001 in. However, pin-to-pin in-plane expansion measurements are only able to capture strains that occur after the gage points are installed in the concrete surface after initial (baseline) measurements are made. This makes pin-to-pin in-plane expansion measurements ideal for monitoring changes in strain, but they require the use of other measurements (such as CI in-plane expansion) to approximate the strains in the concrete prior to the baseline in-plane expansion measurements. Similar to CI measurements, pin-to-pin expansion measurements only define strains at the surface of the concrete and could be influenced by cracks or deformation from loads other than ASR unless these effects are excluded from the measurement.

Total in-plane expansion can be determined by combining expansion up to installation of the pins from CI measurements with the change in expansion from the pin-to-pin expansion measurements.

3. LOADS AND LOAD COMBINATIONS

The original design loads are defined in the UFSAR [12] and SD-66 [13], which provides the structural design criteria for Seabrook Station structures. Evaluation of Seismic Category I structures shall consider all original design loads plus loads due to ASR. Other self-straining loads, including creep, shrinkage, and swelling, shall also be considered, if significant. When the deformed shape due to sustained dead load is small, then the creep effect that is proportional to this deformation is also small. The swelling can be considered to be small when the waterproofing membrane effectively stops intrusion of water into the structure. Evaluation for the original construction loads is not required since the buildings have already been built.

ASR loads, including ASR expansion of structural components and expansion of concrete backfill, are discussed in Section 3.1. Load factors and load combinations including ASR loads are provided in Section 3.2. Other self-straining loads, including creep, shrinkage, and swelling, are discussed in Section 3.3.

3.1 ASR Loads

3.1.1 ASR Expansion of Structural Components

Demands associated with internal ASR expansion shall be applied to structural components as strain loads based on CI measurements and supplemented by pin-to-pin in-plane expansion measurements, if available. The demands associated with internal ASR expansion shall be applied uniformly through the cross-sectional thickness of the structural components (walls, slabs, foundations, etc.) unless otherwise justified. An average strain value may be used when ASR measurements are available on opposite sides of a structural component.

In the MPR/FSEL test programs, control over the loads imposed on the test specimens allowed isolation of ASR as the primary driver for expansion. At Seabrook Station, the cracking condition may be complex due to phenomena and loads other than ASR expansion such as:

- Concrete shrinkage,
- Thermal effects related to high temperatures during the initial concrete placement or during plant operation,
- Construction or equipment loads,

- Pressurization tests of the Containment Structure,
- Structural cracks not indicative of ASR cracking that are defined in Table 1,
- Structural cracks due to differential internal ASR expansions in different regions (see Figure 1 for example), and
- Structural cracks due to externally applied loads from concrete backfill expansion causing differential movements between adjacent buildings or regions of buildings (see Figures 2 and 3 for example).

Large CI measurement values therefore may not necessarily imply large ASR expansions. If some or all of the cracks at an ASR monitoring grid are shown to be caused by a mechanism other than internal ASR in the reinforced concrete member (such as crack types listed above), then the CI value or pin-to-pin expansion measurements should be adjusted accordingly. The adjusted CI value and pin-to-pin expansion shall be computed by excluding the widths of cracks determined not to be caused by ASR. This determination may be through petrography, detailed examination of cracking features, review of operating conditions and history, analytical evaluation for causes other than ASR (e.g. thermal, shrinkage), or other justified means. ASR loads shall be applied to Seismic Category I structures with concrete affected by ASR as described in the following subsections.

3.1.1.1 Containment Structure

ASR in the Containment Structure shall be categorized into Severity Zones based on the magnitude of ASR, as defined in Table 2. CI values supplemented by pin-to-pin measurements (if available) that are used for severity zone categorization should be adjusted to exclude cracking due to repeated pressure testing of the Containment Structure. Although present cracking features may conform to some of the Table 1 definitions, review of the historical crack mapping after pressurization is important to better define the ASR cracking level. The magnitude of the applied ASR strain loads shall be based on the Severity Zone and the analysis stage, as provided in Table 3 with consideration of using bounding values.

3.1.1.2 Containment Internal Structures

For Containment Internal Structures (CIS), the CI values supplemented by pin-to-pin measurements (if available) shall be used for regions identified as having ASR expansion. The

ASR in-plane expansion shall be used to calculate ASR demands for regions impacted by ASR (if any).

3.1.1.3 Other Seismic Category I Structures

For Seismic Category I structures other than the Containment Structure, the CI value supplemented by pin-to-pin measurements (if available) recorded within a particular region shall be used to compute the ASR strain loads from expansion of the structure for that particular region. ASR in-plane expansion can be adjusted when justified to exclude cracks determined not be caused by ASR as discussed in Section 3.1.1. If multiple measurement grids are located within a region, an average measurement value shall be used to compute the ASR strain loads for that region.

3.1.1.4 Foundations for Seismic Category I Structures

Similar to Seismic Category I structures other than the Containment Structure, foundations for Seismic Category I structures shall use the CI value supplemented by pin-to-pin measurements (if available) to represent the ASR strain loads, above. ASR in-plane expansion can be adjusted when justified to exclude cracks determined not be caused by ASR as discussed in Section 3.1.1. If multiple measurement grids are located within a region, an average measurement value shall be used to compute the ASR strain loads for that region. For regions that are not accessible for making field measurements or observations, bounding ASR strain loads shall be considered using available values from other regions within the foundation or within the structure in close proximity to the foundation.

3.1.2 ASR Expansion of Concrete Backfill

ASR expansion of the concrete backfill can create an external pressure on the walls and slabs of structures and lead to structural deformation, rigid body displacement of structures, and relative displacements between adjacent structures. The ASR expansion of the concrete backfill cannot easily be measured directly. External pressure on the walls and slabs due to concrete backfill ASR expansion shall be determined indirectly through field measurements of structural displacements and deformations, and/or through the use of conservative assumptions, as discussed in more detail in Section 4.4.3.2. The magnitude of concrete backfill pressure can be adjusted by correlating the structural analysis deformation under in-situ conditions to field observations.

3.2 ASR Load Factors and Load Combinations

Load factors for ASR loads are based on the work presented in [14]. These load factors were developed to maintain the level of structural reliability provided by the original codes of record load combinations. ASR load factors and load combinations shall be used to compute demands for Seismic Category I structures with concrete affected by ASR as described in the following subsections.

3.2.1 Containment Structure

Load combinations for evaluation of the Containment Structure are provided in Table 4. This is the same table provided in the UFSAR (Table 3.8-1) but with revision to include load factors for ASR loading. The Containment Structure is designed using the ASME Boiler and Pressure Vessel Code [6], which generally implements a working stress design approach and uses load factors of 1.0 for service-level load combinations. Many loads, including ASR, are assigned a load factor of 1.0; however, load factors greater than 1.0 are used for some loads in severe environmental, extreme environmental, and abnormal load combinations. The Containment Structure is not exposed to earth pressures, wind loads, or snow loads, and therefore does not include such loads in its combinations.

3.2.2 Containment Internal Structures

Load combinations for evaluation of Containment Internal Structures are provided in Table 5. This is the same table provided in the UFSAR (Table 3.8-14) but with revision to include load factors for ASR loading. The Containment Internal Structures were designed using the strength design provisions of ACI 318-71, which uses load factors that are greater than 1.0 for normal load combinations as defined in the UFSAR for reinforced concrete structures. Load factors are generally equivalent to 1.0 for unusual load combinations (i.e., load combinations including LOCA, SSE, OBE, high-energy pipe failures, etc.). Containment Internal Structures are not exposed to earth pressures, wind loads, and snow loads, and therefore do not include such loads in their combinations.

3.2.3 Other Seismic Category I Structures

Load combinations for evaluation of Seismic Category I structures other than the Containment Structure are provided in Table 6. This is the same table provided in the UFSAR (Table 3.8-16) but with revision to include load factors for ASR loading. These structures were designed using the strength design provisions of ACI 318-71, which uses load factors that are greater than 1.0 for normal load combinations as defined in the UFSAR for reinforced concrete structures. Load factors are generally equivalent to 1.0 for unusual load combinations (i.e., load combinations including SSE or tornado loads) as defined in the UFSAR.

3.2.4 Foundations for Seismic Category I Structures

Load combinations for evaluation of foundations for Seismic Category I Structures, excluding the foundation of the Containment Structure, are provided in Table 6. These structures were designed using the strength design provisions of ACI 318-71, which uses load factors that are greater than 1.0 for normal load combinations as defined in the UFSAR for reinforced concrete foundations. Load factors are generally equivalent to 1.0 for unusual load combinations as defined in the UFSAR.

Load combinations for evaluation of the Containment Structure foundation are provided in Table 4. The Containment Structure foundation is designed using the ASME Boiler and Pressure Vessel Code [6], which generally implements a working stress design approach and uses load factors of 1.0 for service-level load combinations. Many loads, including ASR, are assigned a load factor of 1.0; however, load factors greater than 1.0 are used for some loads in severe environmental, extreme environmental, and abnormal load combinations.

3.3 Self-Straining Loads other than ASR

Self-straining loads, such as creep, shrinkage, and swelling, shall be considered as dead loads if field observations or analysis has identified that their load effects could be significant.

3.3.1 Creep

Creep due to sustained loads causes additional deflections beyond those which occur when loads are first placed on the structure. Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, quantity of compression reinforcement, magnitude of the sustained load, and other factors [3]. For most cases of long-term deflection in statically determinate structures, the gradual time-change of stresses due to creep is negligible, but time changes of strains may be significant. In statically indeterminate structures, redistribution of internal forces may arise [15]. Creep does not apply any net forces on the reinforced concrete section but instead causes the stresses in the section to redistribute over time. The redistribution from creep causes a portion of the stress in concrete to relax and the stresses in the reinforcement to change proportionally.

The stress redistribution caused by creep can be excluded in the cases described below, when doing so is conservative for the specific conditions under evaluation.

- For sections where sustained loads cause net compression, creep causes a portion of the compression load to shift from the concrete to the reinforcement over time. In such cases, it is conservative to neglect this redistribution because it relieves compression demand in concrete and reduces the tensile demand in the reinforcement.
- For sections where sustained loads cause net tension, creep will cause a portion of the tension to shift from the concrete to the reinforcement over time. This redistribution does not impact this evaluation because ACI 318-71 and ASME already assume that concrete does not resist tension unless a section is specifically designed as a plain concrete section.

The general model for computation of the creep coefficient provided in ACI 209R-92 [15] shall be used.

3.3.2 Shrinkage

Shrinkage is the volume change that occurs during the hardening of concrete and is caused by the loss of water as the concrete cures. Shrinkage strains are independent of the sustained loads acting on a concrete section.

The stresses caused by shrinkage can be excluded from analysis in the cases described below, when doing so is conservative for the specific conditions under evaluation.

- Shrinkage generally acts in a direction opposite to ASR and therefore reduces the demand in sections that are undergoing ASR expansion (provided that the magnitude of ASR expansion is similar to or greater than the shrinkage strain).
- Shrinkage generally causes a small tensile demand in the concrete and a corresponding compression demand in the reinforcement in concrete structures without

significant structural cracking or shrinkage restraint. These stresses are opposite from those typically resisted by the concrete and reinforcement and therefore can be conservatively excluded in most cases.

The general model for computation of concrete shrinkage provided in ACI 209R-92 shall be used.

3.3.3 Swelling

Concrete that is subjected to long-term water exposure exhibits a net increase in volume and mass over time due to swelling. Much like ASR expansion, concrete swelling will generally cause tension in the reinforcement and compression in the concrete.

Research has shown that reinforced concrete under conditions similar to those at Seabrook Station may have swelling expansion of 100×10^{-6} in./in. in the portions that are permanently exposed to groundwater [16].

4. ANALYSIS APPROACH

A three-stage analysis approach shall be used for analyzing Seismic Category I structures, as identified in the Seabrook LAR 16-03. Each stage applies increasingly sophisticated methods and uses additional field data to improve the rigor of the analysis, and without impacting the code inherent safety factors. The analysis of each structure may begin at any stage and, if necessary, shall progress to a higher analysis stage.

4.1 Selection of Starting Stage

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

1. Structures with simple geometry that permits structural analysis using closed-form solutions and/or simple finite element models
2. Structures with localized ASR expansion, or ASR expansion affecting the structure as a whole but with only minor indications of distress
3. Structures with an apparent robust original design leading to a reasonable amount of margin to accommodate ASR demands
4. Structures that do not exhibit significant signs of distress

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

Note that the above process for selection of starting stage is a guideline, and the starting stage may be adjusted after performing preliminary analysis/review. Stage One Screening Evaluation, Stage Two Analytical Evaluation, and Stage Three Detailed analyses are described in following Sections 4.2, 4.3, and 4.4, respectively.

4.2 Stage One Screening Evaluation

Structural evaluation should start at Stage One Screening Evaluation if the structure meets all four criteria listed in Section 4.1.

4.2.1 Field Observations to Support Stage One Analyses

Walkdowns of structures and plant equipment have been performed by NextEra to identify symptoms of ASR presence. Inspection data from these walkdowns shall be used in conjunction with other previous measurements to identify potential locations and directions of structural movement and deformation. The previous data include measurements of relative building movements, equipment misalignments, and CI. Indications of deformation or ASR conditions documented in NextEra Action Requests (ARs), and Work Orders (WOs) shall be reviewed.

After reviewing previous field data, a walkthrough inspection shall be performed to verify field conditions and determine whether ASR expansion only affects localized regions of the structure or whether the structure has experienced global deformations. Previous field data that are older than three years shall be verified during this walkthrough inspection. Additional field measurements can be taken during these walkthrough inspections, but collection of extensive field data is not intended for this inspection.

Many of the existing ASR monitoring grids for measurement of CI that have been installed prior to the start of ASR susceptibility evaluation are at reasonably-accessible locations of most apparent ASR cracking identified by visual inspection. These measurements, when used in combination with pin-to-pin in-plane expansion measurements, provide a conservative estimate of the ASR strain in the structure. If a new ASR monitoring grid must be installed to support a Stage One analysis, it shall be installed at the location where cracking is most apparent, based on visual observation, within the region that requires additional data. Measurement data used for Stage One analysis shall be from measurements made within three years, in accordance with the Seabrook LAR 16-03 [1]. Monitoring requirements are listed in Table 7.

4.2.2 Non-ASR Demands for Stage One Analyses

The demands from the original design load calculations (non-ASR demands) shall be used. If required, when demands are either not calculated in the original design or the calculated demands are overly conservative, the non-ASR demands may be recalculated using methods that are generally consistent with the original design methodology.

Effects of creep, shrinkage, and swelling are considered small in the Stage One analysis methodology; therefore, structures being evaluated in this stage should have little to no sign of distress due to these self-straining loads.

4.2.3 ASR Demands for Stage One Analyses

Structural demands caused by ASR loads shall be computed by using conservative and simplified structural analysis methods similar to the original design calculations, or using simple finite element models. Analysis shall consider the structure to be subject to ASR expansion of structural components on a conservative basis using the limited but conservative field inspection data. For structures embedded in concrete backfill, analysis shall also consider pressure on the structure due to concrete backfill ASR expansion.

4.2.3.1 ASR Expansion of Structural Components

Regions of the structure that exhibit features typical of ASR shall be analyzed for ASR expansion corresponding to the field measurement value obtained from one of the most severe cracking locations within the region. ASR loads shall be applied as uniform strain through the concrete thickness of the structural component to cause an expansion that is consistent with the assigned ASR expansion value or the assigned bounding value for the case of the Containment Structure as discussed in Subsection 3.1.1.1.

4.2.3.2 ASR Expansion of Concrete Backfill

For structures embedded in concrete backfill without an isolation gap, the structure and the backfill shall be considered to be in direct contact, even though shrinkage of the concrete backfill and structure and possible deterioration of the waterproofing backboard could result in a separation.

Pressures acting laterally on walls due to concrete backfill ASR expansion shall be taken as equal to the overburden pressure at the elevation under consideration. The overburden pressure is considered to be the approximate upper-limit for the unfactored lateral pressure. Once this pressure is reached, additional ASR expansion will occur preferentially in the vertical and/or other transverse directions. Lower concrete backfill pressure can be used when it is justified based on structural deformation or lack of distressed area by field observations.

Concrete backfill ASR pressures acting laterally on walls shall be determined by following the steps shown schematically in Figure 4.

4.2.3.3 Use of Cracked Section Properties in Stage One Analysis

Stage One analysis should generally use uncracked section properties for conservatism. Cracked section properties, however, may be used to account for the effects of structural cracking on flexural stiffness of a member provided both of the following conditions are met:

- Cracked section properties were used in the original design calculations.
- Using cracked section properties to represent flexural stiffness for calculating ASR demands would not affect the non-ASR demands calculated in the original design documents.

The flexural stiffness of members with structural cracks, or members expected to develop structural cracks, reduces compared to uncracked section properties. It should be noted that the structural cracks are those due to external loading and not microcracks associated with ASR behavior. The ratio of the cracked to uncracked moment of inertia can be calculated using Equation 9-4 of ACI 318-71 or set equal to 50% of the gross moment of inertia, per Supplement 4 to ACI 318-71 discussed in Section 5.6.

4.3 Stage Two Analytical Evaluation

Structures should start at Stage Two Analytical Evaluation if they meet two or three criteria listed in Section 4.1. Structures that do not meet acceptance criteria using the conservative methods of the Stage One analysis can also be evaluated using Stage Two analysis.

Effects of creep, shrinkage, and swelling are considered small in the Stage Two analysis methodology; therefore, structures being evaluated in this stage should show little to no sign of distress due to these self-straining loads.

4.3.1 Field Observations to Support Stage Two Analyses

Stage Two analyses are supported by additional structural inspections and field measurements beyond those already performed for a Stage One analysis to provide a broader and more accurate assessment of ASR effects on the structure. Field observation may also require identifying structural distress areas (if any) and quantifying the structural deformations to correlate analysis results to field observations. Measurement data used for Stage Two analysis shall be from measurements made within the past 18 months, in accordance with the Seabrook LAR 16-03 [1]. Monitoring requirements are listed in Table 7.

4.3.2 Non-ASR Demands for Stage Two Analyses

The demands from the original design load calculations (non-ASR demands) shall be used. If the required demands are not calculated in the original design or if the calculated demands are overly conservative, then the demands may be recalculated using methods that are generally consistent with the original design methodology.

4.3.3 ASR Demands for Stage Two Analyses

Structural demands caused by ASR loads shall be computed by performing finite element analysis of the structure subject to ASR expansion for regions showing ASR expansion. For structures embedded in concrete backfill, analysis shall also consider pressure on the structure due to concrete backfill ASR expansion.

4.3.3.1 ASR Expansion of Structural Components

Regions of the structure that exhibit features indicative of ASR shall be analyzed for ASR expansion corresponding to the average field measurement value from measurements made within the region. ASR loads shall be applied as uniform strain through the concrete thickness of the structural component within each region to cause an expansion that is consistent with the assigned ASR expansion value as discussed in Subsection 3.1.1.

Structural walls, for which the concrete is generally modeled in finite element analysis using shell elements, shall be subject to uniform imposed load through-thickness strains to cause an expansion that is consistent with field measurements of ASR in-plane strain. In these locations where ASR expansion is applied, steel reinforcement shall be also be modeled, and in the case of structural walls the reinforcement should be modeled using membrane elements with orthotropic properties to account for different quantities of steel in horizontal and vertical directions. Modeling of steel reinforcement shall take into consideration the development length required to be fully effective. Testing has shown that the adequacy of reinforcement development length is not impacted by ASR for the expansion up to the limit defined in

MPR-4273 [4]. In-plane gradients in ASR expansion may be used to transition between regions of different ASR expansion quantities. While the description above is specific to reinforced concrete structural walls modeled with a combination of shell and membrane elements, this methodology may be extended to other type of elements (beam, solid, etc.).

4.3.3.2 ASR Expansion of Concrete Backfill

For structures embedded in concrete backfill without an isolation gap, the structure and the backfill shall be considered to be in direct contact, even though shrinkage of the concrete backfill and structure, and possible deterioration of the waterproofing backboard, could result in a separation.

Concrete backfill ASR pressures acting laterally on walls shall be taken as equal to the overburden pressure at the elevation under consideration. The overburden pressure is considered to be the approximate upper-limit for the unfactored lateral pressure. Once this pressure is reached, additional ASR expansion will preferentially occur in the vertical and/or other transverse directions.

If field observations of the wall show no signs of distress, then the unfactored backfill pressure may be reduced and be limited to the pressure that would initiate observable distress in the wall. In most cases, distress would be observable as flexural cracking. Concrete backfill ASR pressures acting laterally on walls shall be determined by following the steps shown schematically in Figure 4.

4.3.3.3 Use of Cracked Section Properties in Stage Two Analyses

Stage Two analysis may use cracked section properties as explained in Section 4.4.5 for Stage Three analysis provided that the use of cracked section properties does not invalidate the methodology used in the original design calculation or change the load path for the original design loads.

For Stage Two evaluation, the iterative steps that are explained for Stage Three evaluation may be stopped after a single iteration (initial step plus iterative step 1) because the demands computed would be conservative as explained in Section 4.4.5.

4.4 Stage Three Detailed Evaluation

Structures should start at Stage Three if they meet one or none of the criteria listed in Section 4.1. Structures that do not meet acceptance criteria using the conservative method of the Stage Two analysis shall be evaluated using Stage Three analysis.

4.4.1 Field Observations to Support Stage Three Analyses

Stage Three analyses may need to be supported by additional inspections beyond those already performed for Stage Two to provide a broader and more accurate assessment of ASR effects on the structure. Structural deformation also shall be measured at sufficient locations for estimating deformed geometry of the structure or specific structural member(s). Field observation should also identify locations with structural distress, such as structural cracking and/or concrete cover delamination (if any), so that correlations can be made between the analytical results for the in-situ condition and field observations; this is discussed in more detail in Section 4.4.4. Measurement data used for Stage Three analysis shall be from measurements made within the past 6 months, in accordance with the Seabrook LAR 16-03 [1]. Monitoring requirements are listed in Table 7.

4.4.2 Non-ASR Demands for Stage Three Analyses

Finite element analysis shall be used to compute the non-ASR structural demand forces more rigorously than in the original design calculation. The non-ASR loads (i.e. - wind, seismic, hydrostatic pressure, etc.) from the original design calculations shall be applied to the finite element model unless otherwise justified. For example, reduced loads may be applied to the model if the original design used loads that enveloped both Unit 1 and Unit 2 conditions but the loading is shown to be conservative for Unit 1 which is of interest. Another justified circumstance is when the original design utilized conservative preliminary loading prior to defining the actual loading (e.g., equipment loading and thermal loading). The original design generally used simplified analysis methods or models to calculate structural responses, while the detailed finite element model used in Stage Three analysis more rigorously calculates the structural demands. Comparison between the design loads determined from the Stage Three finite element analysis and original design calculations should be made, and reason(s) for differences should be determined.

For seismic loads, the maximum seismic acceleration profiles in each direction (typically N-S, E-W, and vertical) provided in the original design calculations shall be applied to the finite element model independently. Because elements in Stage Three analytical models are typically more discretized than in the original design models, linear interpolation may be used to obtain the maximum acceleration at each node of the Stage Three finite element model. The seismic mass of the structure shall include all dead weight including fixed equipment, piping, and etc. The response of each directional input shall be combined using the SRSS method in accordance with Regulatory Guide 1.92 [18].

Creep, shrinkage, and swelling shall be considered in Stage Three analysis when comparing simulated deformations to field measurements if field observations or analysis has determined that their load effects could be significant.

4.4.3 ASR Demands for Stage Three Analyses

Structural demands caused by ASR loads shall be computed by performing finite element analysis of the structure subject to ASR expansion for regions affected by ASR. For structures embedded in concrete backfill, analysis shall also consider pressure on the structure due to concrete backfill ASR expansion. ASR expansions of the structural concrete and backfill concrete will produce stresses in one another. The interaction between ASR-induced expansion of the structure and stiffness of concrete fill restraining the expansion of the structure shall be considered as described in following subsections.

4.4.3.1 ASR Expansion of Structural Components

Structural members with significantly varying CI values will be divided into regions to better define the in-plane expansion variation for structural members. Each region of the structure that exhibits features indicative of ASR shall be assigned an average field measurement value from measurements made within that region. ASR loads shall be applied as uniform strain through the concrete thickness of the structural component to cause an expansion that is consistent with the assigned ASR expansion value as discussed in Subsection 3.1.1.

Structural walls, which are generally modeled in finite element analysis using shell elements or beam elements, shall be subject to uniform strain through the thickness to cause an expansion in the concrete model that is consistent with field measurements of ASR in-plane strain. In locations where ASR expansion is applied, steel reinforcement shall be also be modeled, and in the case of structural walls and slabs the reinforcement should be modeled using membrane elements with orthotropic properties to account for different quantities of steel in horizontal and vertical directions. Modeling of steel reinforcement shall take into consideration the development length required to be fully effective. Testing has shown that the performance of reinforcement development length is not impacted by ASR for the expansions up to the limit defined in MPR-4273 [4]. In-plane gradients in ASR expansion may be used to transition between regions of different ASR expansion quantities. While the description above is specific to reinforced concrete structural walls modeled with a combination of shell and membrane elements, this methodology may be extended to other types of elements (beam, solid, etc.).

In locations where concrete backfill is adjacent to structural components without a seismic isolation gap, the stiffness of the concrete backfill shall be accounted for in the finite element model. The stiffness of the backfill elements shall be determined as follows:

- If the concrete backfill has sufficient stiffness to fully restrain a structural component (e.g., wall or slab) that is affected by ASR, then the total member strain will be limited due to the restraint. This restraint can be observed by reviewing if the pin-to-pin expansion measurement over time remained constant or increased slightly. The effective stiffness of the backfill concrete in these cases shall be quantified using the elastic modulus from the following equation.

  $$E_{backfill} = E_C k_{r1}$$

  E_c is the elastic modulus of backfill concrete per Section 8.3 of ACI 318-71 based on the original design specified compressive strength.

  k_{r1} is a knockdown factor that represents the reduction in elastic modulus of unreinforced concrete due to ASR. Material testing indicates that a reduction in elastic modulus occurs in unreinforced concrete with ASR [19], [20], [7], [11], and [21]. In Stage Three analyses, a 70% stiffness reduction shall be used for backfill concrete with 0.35% expansion or higher, and the stiffness reduction shall decrease proportionally to 0% at zero expansion. This stiffness reduction is based on data reported in various sources ([19], [20], and [7]). The strain of concrete fill can be estimated based on measured CI expansion measured in adjacent structural members or observed structural deformation relative to the width of the concrete fill perpendicular to the plane of the below-grade wall.

- If pin-to-pin in-plane expansion measurements indicate that the total strain of the component is increasing over time, then the backfill must have a reduced stiffness to accommodate this strain. The stiffness of the backfill shall be quantified using the elastic modulus from the equation below.

$$E_{backfill} = E_c k_{r1} k_{r2}$$

k_{r2} is a knockdown factor that represents additional reduction in backfill concrete elastic modulus due to shrinkage, compression of waterproofing backboard, and/or crushing. The k_{r2} knockdown factor shall only be used when field measurements indicate that backfill concrete does not restrain the ASR expansion of a structural component as much as analysis using the stiffness $E_c k_{r1}$ would indicate. The value of k_{r2} shall be determined empirically through finite element model simulations that compare the in-situ conditions of the structural component of interest, including the adjacent members, to field observations and to locations of distressed areas or lack of observed distressed areas.

4.4.3.2 ASR Expansion of Concrete Backfill

For structures embedded in concrete backfill without an isolation gap, the structure and the backfill shall be considered to be in contact, even though shrinkage of the concrete backfill and structure, and possible deterioration of the waterproofing backboard, would result in a separation. Concrete backfill ASR pressures acting laterally on walls shall be determined by following the steps described below and shown schematically in Figure 4.

- **Step One:** The maximum unfactored lateral pressure on walls from concrete backfill ASR expansion shall be taken as equal to the overburden pressure at the elevation under consideration.

- **Step Two:** If structural deformation measurements associated with backfill pressure are available, then the backfill pressure may be limited to the pressure that simulates the measured deformations. The simulated deformations calculated based on in-situ loading consist of unfactored sustained loads and unfactored ASR expansion of the structural components; therefore, limiting ASR pressure from concrete backfill should simulate the measured deformations. This limit may only be imposed if an assessment determines that the deformation measurements were, in part, caused by out-of-plane pressures imposed by ASR expansion of the concrete backfill.

- **Step Three:** If field observations of the wall show no signs of distress, such as flexural cracking, then the unfactored backfill pressure may be limited to the pressure that would initiate observable distress in the wall.

- **Step Four:** If field observations of the wall show signs of distress associated with concrete backfill pressure, and if the distress is a symptom of a ductile load effect (such as flexure), then the unfactored backfill pressure may be limited to the pressure that would cause the observed level of distress. Computation of this limiting pressure requires sufficient measurements to quantify the level of distress, and sufficiently detailed simulation to correlate the level of distress with concrete backfill pressure. The simulation shall consider unfactored sustained loads and unfactored ASR expansion of the structure acting in addition to the backfill pressure. Since distress is observed in the in-situ condition, stiffness reductions due to structural cracking in accordance with Section 4.4.5 may be used when simulating the backfill behavior.

Note: Step four may only be used in Stage Three analyses.

- **Step Five:** When the interfacial compressive stress between the concrete backfill and the structural wall reaches or exceeds the overburden compression stress, further expansion of the concrete backfill will occur preferentially in the vertical and/or other transverse directions. Therefore, the interfacial compressive stress determined from analysis with ASR expansion of structural components alone calculated per Section 4.4.3.1 can be subtracted from the pressure determined from Step 1.

  Note: Step five may only be used in Stage Three analyses.

- **Step Six:** The unfactored pressure exerted on the structure due to concrete backfill ASR expansion, p_H, shall be computed as:

$$p_H = \min[p_{H2}, p_{H3}, p_{H4}, (p_{H1} - p_{H0})] \geq 0\, psi$$

  Where p_{H0} is the compressive stress applied to the concrete backfill by ASR expansion of the structural components calculated per Section 4.4.3.1, and p_{H1}, p_{H2}, p_{H3}, and p_{H4} are limiting backfill pressures from Steps 1, 2, 3, and 4, respectively.

  In this equation, the overburden pressure is reduced by the compressive stress in the backfill due to other loads (expressed as $(p_{H1} - p_{H0})$). This is done to avoid double-counting of the pressures acting on the interface between the backfill and structure.

  Note: If the criteria for the limiting pressure p_{H2}, p_{H3}, or p_{H4} are not met, then the associated limiting pressures shall be excluded from the equation above.

- **Step Seven:** For factored load conditions, p_H shall be amplified by the product of the load factor for ASR and the threshold factor as defined in Section 6. In lieu of applying concrete backfill pressure for cases where it would produce unrealistic deformation (due to structural cracking or structural movement), the concrete backfill ASR expansion (deformation) can be amplified by the product of the load factor for ASR and the threshold factor while considering increased cracking in the restraining structure.

Research has shown that ASR expansion occurs preferentially in directions with less compression stress [4], [19], [22], and [21]. In the above methodology, the overburden pressure is treated as an approximate upper-limit for the unfactored lateral pressure. Once this pressure is reached, additional ASR expansion will occur preferentially in the vertical and/or other transverse directions. When consideration is given to the load factor for ASR expansion, an out-of-plane pressure of up to twice the overburden pressure (load factor for static load combination) may be applied laterally to walls. This factor of 2 for pressure may be reduced, but not to less than the load factor for dead load when the field data confirms the rigid body vertical movement of the structure components such as base slabs that overlay the concrete backfill.

Overburden pressures include the weight of structures, soil, permanent equipment, and concrete backfill above a certain elevation. Overburden pressures can also include clamping pressures imposed by walls that are tensioned by vertical concrete backfill expansion, such as those shown in Figures 2 and 3. When concrete backfill is in a narrow region, such as that shown in Figure 2b, analysis may show that the overburden stress is partially reduced by the adjacent rock.

Concrete backfill ASR vertical expansion that creates uplift pressure below a base slab shall be determined by the following steps:

- **Step One:** Determine if the concrete backfill is in a confined region, such as those illustrated in Figures 2 and 3. When a confined region exists, vertical expansion of the fill also causes an axial tension in the confining walls in addition to uplift pressure on the base slab.

- **Step Two:** If the concrete backfill is in a confined region, then the expansion of the fill shall be modeled using a vertical-upward pressure acting on the base slab. When field measurements of vertical displacement and/or vertical strain in the connecting wall(s) are available, the magnitude of the upward pressure shall be adjusted to simulate measurements. The upward pressure is a function of the stiffness of the structure, which shall be adjusted in accordance with Section 4.4.5 to account for cracking. For factored load conditions, the upward pressure shall be amplified by the product of the ASR load factor and the threshold factor as defined in Section 6. Alternatively when the restraining structure is cracking, the vertical displacement/strain shall be amplified by the product of the ASR load factor and threshold factor, and then the upward pressure associated with this amplified displacement/strain shall be treated as the factored pressure.

- **Step Three:** If the concrete backfill is not in a confined region, then vertical expansion of the fill may cause upward rigid body movement of the structure with some possible rigid-body rotation. If there are discontinuities in the upward movement, then demands resembling differential settlement may occur; such effects shall be evaluated when apparent. As described in Section 5, the impact of ASR on global stability shall be assessed.

4.4.4 Correlation of Analysis Model to Field Observations

The finite element analysis results for in-situ conditions should correlate with the deformations, strains, and distressed areas (if any) observed onsite. Further refinement of modeling procedures or additional field observations may be required to improve the correlation between analysis results and field observations for locations and types of distress (such as crack type, crack direction, location of cracking region, etc.) and deformation. Deviations between analytical results and field observations could be from incorrect modeling assumptions, ASR load, ASR modeling steps discussed in Section 4.4.3, or other self-straining load that must be adjusted to improve the correlation between the finite element results and field observations. Considering cracked section properties, modifying the boundary conditions, and adjusting the ASR expansion pressure for concrete backfill, when justified, can be used to improve the analysis model. The analysis model is considered acceptable when the following conditions are consistent with field observations:

- Location of major structural cracks, as well as the type and direction of structural cracking regions,
- Structural deformation patterns and locations and magnitudes of critical deformation within the accuracy of the measurements and structural construction tolerances, and
- Relative movement between adjacent structures or between structures and components or piping at critical locations.

4.4.5 Refined Analytical Methods

The effect of structural cracking on reducing the axial, shear, and flexural stiffness of a structural component may be considered if at least one of the following conditions is met:

- Field investigation indicates the formation of structural cracks, i.e. - flexural cracks, membrane/axial cracks, and/or shear cracks.
- Flexural cracks are expected to be accompanied by out-of-plane movement.
- Axial/membrane cracks are expected to form in the direction perpendicular to tensile stress direction.
- In-plane shear cracks are expected to be inclined to the primary directions of reinforcement, e.g., 45°.
- Finite element analysis shows that the out-of-plane bending moment of a shell element (representing a wall or slab) or moment about the principal axis of a beam element (representing a beam or column) exceeds the flexural cracking moment, where the magnitude of cracking moment shall be computed using Equation 9-5 of ACI 318-71 as:

$$M_{cr} = \frac{f_r I_g}{y_t}$$

where f_r is the concrete modulus of rupture, I_g is the gross moment of inertia, and y_t is the distance between the extreme tensile fiber and the centerline of the cross-section. The value of f_r can be computed using Equation 9-5 of ACI 318-71 as:

$$f_r = 7.5\sqrt{f_c'}$$

where f_c' is the specified compressive strength of the concrete.

- Finite element analysis shows that the 1st principal mechanical strain of an element (shell, solid or beam element) exceeds the cracking strain computed from the following equation:

$$\varepsilon_{cr} = f_t/E_c$$

where E_c is the elastic modulus of concrete and f_t is the uniaxial tensile strength of concrete material. The value of f_t can be computed using the following equation which is within the value range recommended by [23]:

$$f_t = 5\sqrt{f_c'}$$

Flexural cracks are common in concrete structures, and are allowed to form by the code of record [3]. Cracked section properties due to flexural cracks were also used in the original design calculations of the structures at Seabrook Station.

Axial cracking caused by ASR expansion of structural components usually occurs at the intersection of two components or at boundaries of two regions with different magnitudes of ASR expansion. Any difference in ASR expansion between two regions will subject the region with higher ASR to compression while causing tension in the other region. This phenomenon is schematically depicted in Figure 1 for a sample wall-to-slab connection. Another example is the wall that connects two separate structural slabs with relative upward motion due to expansion of concrete backfill within a confined region as depicted in Figure 3.

To account for the effects of cracking, the axial rigidity (EA), shear rigidity (GA), and/or flexural rigidity (EI) of the cracked element must be reduced. In finite element modeling, the reduction cannot be applied merely to the E (modulus of elasticity) and/or G (shear modulus), because any reduction in E will affect both axial and flexural rigidities, i.e. - they are coupled. The reduction in axial rigidity should be applied only in the direction perpendicular to the cracks.

Additionally, flexural cracks can form without the formation of axial cracks, or axial cracks can form only in one direction; therefore, it is necessary to formulate a decoupled set of equations that allow independent reduction in axial, shear, and flexural rigidities. This requires the use of orthotropic material properties combined with modified cross-sectional dimensions, as explained in the step-by-step procedure provided in Appendix A.

5. ACCEPTANCE CRITERIA

The effect of ASR on the structural design basis of affected concrete structures at Seabrook Station is evaluated in MPR Report MPR-4288 [5]. It assesses the impact of ASR on structural limit states (flexure, shear, and compression capacities and that of attachments to concrete structures) as well as several additional design considerations. For the limit states of shear, flexure, and reinforcement development and lap length, results from the MPR/FSEL test programs suggest that the effects of confinement from the reinforcement more than compensate for degradation of material properties when ASR progression is within the range observed in the test specimens [5]. MPR-4288 concludes that the effects of ASR expansion on the structural behavior of reinforced concrete structures can be explained with basic structural mechanics and that these effects can be evaluated using the provisions of the structural design codes applicable to Seabrook Station. Based on this conclusion, acceptance criteria from the applicable Codes of Record, supplemented or modified as described in this document, shall be applied for meeting the intent of the Codes. Technical justification for the application of acceptance criteria not specified in the Codes of Record is provided elsewhere in this document and is based on structural mechanics and sound engineering practices consistent with the Codes of Record.

Acceptance criteria shall be calculated using material properties specified in the original design basis. The basis for the use of design material properties is provided in MPR-4288. The applicability of the conclusions provided in MPR-4288 and used herein is based on the large scale test program described in MPR-4273 [4] and is limited to the extent of ASR expansion documented in MPR-4273. Accordingly, for analyses of concrete structures at Seabrook Station, the elastic modulus of concrete shall be computed using the minimum design compressive strength, and no reduction shall be taken for ASR-related damage. Material properties for analysis and evaluation of Seismic Category I structures shall be consistent with those used in original design calculations and those defined in the project specifications and drawing notes.

Structural acceptance criteria for evaluation of Seismic Category I structures with concrete affected by ASR shall be in accordance with the applicable Codes of Record except for deviations listed in Section 5.6, which are considered as supplements to the Codes of Record.

A summary of the acceptance criteria as defined in the UFSAR is provided in the following sections.

5.1 Containment Structure

Structural acceptance criteria for evaluation of the Containment Structure shall be in accordance with UFSAR Section 3.8.1.5.

5.1.1 General

The Containment Structure, including liner and penetrations, shall remain within elastic limits under service load conditions and under the mechanical loads of the factored load conditions. With thermal loads included, the reinforcing steel in some regions may yield, but the strain shall not exceed twice the yield strain. Gross deformations of the Containment Structure shall not cause the structure to contact other structures or components. Service load combinations include conditions encountered during testing, normal operation, shutdown, and severe environmental conditions; these are listed in Table 4. Factored load combinations include those conditions resulting from severe environmental, extreme environmental, abnormal, abnormal/severe environmental, and abnormal/extreme environmental loads, as defined in ASME and listed in Table 4.

The design limits imposed on the various parameters that serve to quantify the structural behavior and provide a margin of safety shall be in compliance with ASME Sec. III, Div. 2. The allowable limits on these parameters, for service and factored loads, are given in Table 3.8-2 of the UFSAR.

5.1.2 Concrete

The allowable compressive stresses, including membrane, membrane plus bending and localized stresses, and shear stresses under service loads and factored loads are as specified in ASME Sec. III, Div. 2, Article CC-3400, with the exceptions related to shear stresses as specified in UFSAR Section 3.8.1.5 (b).

5.1.3 Reinforcing Steel

The stress and strain limits for reinforcing steel under service and factored loads are as specified in ASME Sec. III, Div. 2, Articles CC-3432 and CC-3422, respectively.

If local yielding occurs under combined mechanical and thermal load, the net strain shall be less than twice the yield strain, as established in the 1977 Winter Addendum to ASME Sec. III, Div. 2, Article CC-3422.1 (d). This strain limit insures that the yielding under thermal load does not result in concrete cracking which would cause deterioration of the Containment Structure.

5.1.4 Liner Plate and Liner Anchorage System

The evaluation of the liner plate and its anchorage system is not part of structural ASR susceptibility analysis described in this methodology document.

5.1.5 Stability

Acceptance criteria for stability against overturning, sliding, and flotation are as defined in Table 8.

5.2 Containment Internal Structures

Structural acceptance criteria for evaluation of Containment Internal Structures shall be in accordance with UFSAR Section 3.8.3.5. The basis for the development of the stress-strain criteria are the ACI 318-71 code. The reinforced concrete structures must meet ACI 318-71 with supplements as listed in Section 5.6.

5.2.1 Normal Load Conditions

Normal load conditions are those encountered during testing and normal operation as defined in UFSAR Subsection 3.8.3.3. They include dead load, live load, ASR load, and anticipated transients or test conditions during normal and emergency startup and shutdown of the Nuclear Steam Supply, Safety, and Auxiliary Systems. Normal loading also includes the effect of an Operating Basis Earthquake.

5.2.2 Unusual Load Conditions

Unusual load conditions are those conditions resulting from combinations of the LOCA, SSE and OBE, high-energy pipe failures, ASR, and live and dead loads as defined in UFSAR Subsection 3.8.3.3. The upper bound of elastic behavior is taken as the yield strength capacity of the load carrying components. The yield strength of structural and reinforcing steel is to be taken as the minimum guaranteed yield stress as given in the appropriate ASTM Specification.

5.2.3 Deformations

The deformation for each of the structures due to ASR (if any) is to remain small so that no gross deformations will occur and cause contact with other structures or pieces of equipment.

5.3 Other Seismic Category I Structures

Structural acceptance criteria for evaluation of other Seismic Category I concrete structures shall be in accordance with UFSAR section 3.8.4.5. The basis for the acceptance criteria is the ACI 318-71 Code. However, under the action of seismic or wind loadings, in accordance with the standard review plan (Section II.5), the 33 percent increase in allowable stresses is not permitted. The reinforced concrete structures must meet ACI 318-71 with supplements as listed in Section 5.6.

5.3.1 Normal Load Conditions

Normal load conditions are those encountered during testing and normal operation as defined in UFSAR Subsection 3.8.4.3 and are referred to in the standard review plan as service load conditions. They include dead load, live load, ASR load and anticipated transients, loads occurring during normal startup and shutdown, and loads occurring during emergency shutdown of the nuclear steam supply, safety, and auxiliary systems. Normal loading also includes the effect of an operating basis earthquake and normal wind load.

5.3.2 Unusual Load Conditions

The unusual load conditions include ASR load and those loads shown in Subsection 3.8.4.3 of UFSAR. The upper bound of elastic behavior is to be taken as the yield strength capacity of the load carrying components. The yield strength of reinforcing steel is to be taken as the minimum specified yield stress as given in the appropriate ASTM Specifications.

5.3.3 Deformations

The additional deformation due to ASR for each of the structures shall not cause contact with other structures or pieces of equipment in accordance with UFSAR Section 3.8.4.5.c.

5.3.4 Stability

Acceptance criteria for stability against overturning, sliding and flotation are as defined in Table 8.

5.4 Foundations for Seismic Category I Structures

Structural acceptance criteria for the evaluation of foundations for Seismic Category I structures shall be in accordance with UFSAR Section 3.8.5.4. The acceptance criteria relating to stress, strain, gross deformation, and shear loads are described in Subsections 3.8.1.5 and 3.8.4.5 of the UFSAR for the Containment Structure and other Seismic Category I structure foundations, respectively. Safety factors for buoyancy, sliding, and overturning are given Section 5.1.5:

5.5 Acceptance Criteria for Isolation Gaps

The maximum displacements of adjacent structures due to a design seismic event, when combined with the deformations due to ASR (total ASR deformation including threshold value), shall not exceed the designed gap between these structures. Where the gap can be measured, then the maximum displacements of adjacent structures due to a design seismic event, when combined with the additional ASR threshold deformations, shall not exceed the field measured gap between these structures. The gap between two adjacent structures, A and B, shall satisfy the following equation:

$$\Delta_{Gap} \geq \Delta_{transient} + \Delta_{ASR}$$

Where,

$$\Delta_{\text{transient}} = \sqrt{\Delta^2_{Axis1} + \Delta^2_{Axis2}}$$

$$\Delta_{\text{Axis1}} = \Delta_{\text{Axis1,A}} + \Delta_{\text{Axis1,B}}$$

$$\Delta_{\text{Axis2}} = \Delta_{\text{Axis2,A}} + \Delta_{\text{Axis2,B}}$$

$$\Delta_{ASR} = \Delta_{ASR,A}\left(k_{th,A}k_{Sa,A} - k_0\right) + \Delta_{ASR,B}\left(k_{th,B}k_{Sa,B} - k_0\right)$$

Δ_{Gap} is the design gap between structures A and B when the gap cannot be measured due to accessibility. When the gap can be measured Δ_{GAp} is the field measured gap between structures A and B.

$\Delta_{transient}$ is the movement of the structures due to factored transient loads. The equation for $\Delta_{transient}$ is used in existing evaluations performed by NextEra.

Δ_{Axis1} and Δ_{Axis2} are the absolute sum of the maximum elastically computed displacements in directions 1 and 2 of structures A and B due to factored transient loads. The two directions 1 and 2 are perpendicular horizontal axes (such as north-south and east-west) at the location under evaluation.

$\Delta_{\mathrm{Axis1,A}}$ and $\Delta_{\mathrm{Axis2,A}}$ are structure A displacements due to transient in directions 1 and 2;

$\Delta_{\mathrm{Axis1,B}}$ and $\Delta_{\mathrm{Axis2,B}}$ are structure B displacements due to transient in directions 1 and 2.

Δ_{ASR} is the movement of the structures at the evaluation location in the direction towards each other due to ASR expansion.

$\Delta_{ASR,A}$ and $\Delta_{ASR,B}$ are ASR deformations of structures A and B (based on structural analysis).

$k_{th,A}$ and $k_{th,B}$ are threshold factors associated with structures A and B.

$k_{Sa,A}$ and k_{SaBA} are the load factors for ASR for the load combination under consideration associated with structures A and B.

k_0 is equal to 0 (zero) when the gap can't be measured, and is equal to 1 when the gap is measured.

In the equation for Δ_{ASR}, the product of the threshold factor and load factor for each structure is reduced by $k_0 = 1$ for the condition when the gap width between structures is based on field measurements that already includes $\Delta_{ASR,A} + \Delta_{ASR,B}$. If the reduction in seismic gap width (as measured in the field prior to analysis) exceeds the ASR deformation simulated in analysis at a particular location, then the resulting impact of Δ_{ASR} shall be assessed.

5.6 Supplement to Code of Record Acceptance Criteria

Seismic Category I structures with concrete affected by ASR shall meet the acceptance criteria of the Codes of Record, with the following list of deviations which are considered as supplements to the Codes of Record:

- Supplement 1 - Consideration of ASR loads: The UFSAR load and load combinations Tables 3.8-1, 3.8-14, and 3.8-16 were modified in LAR 16-03 to consider the ASR load and load factors for calculating the total demands on structures affected by ASR.

Basis: ASME 1975 [6] and ACI 318-71 [3] codes do not address the requirement to consider any special loading for reinforced concrete structures that are impacted by ASR behavior. The concrete ASR growth causes stresses in reinforcement and concrete and member forces due to deformation compatibility between members expanding due to ASR. SGH Report 160268-R-01 [14] defines the ASR load and associated load factors to be combined with the original factored load combinations to provide safety margins comparable to those provided by the original factored loads in the original codes of record.

- Supplement 2 – Code acceptance criteria: Strength of reinforced concrete sections affected by ASR can be calculated using the Codes of Record (ASME 1975 and ACI 318-71) and the minimum specified design concrete strength, provided that through-thickness ASR expansion is within the limits stated in report MPR-4273 [4].

  Basis: Report MPR-4288 [5] provides the basis for Supplement 2; the report conclusions are supported by the MPR/FSEL large scale test program described in report MPR-4273.

- Supplement 3 – Shear-friction capacity for members subjected to net compression: The shear-friction capacity for member subjected to net compression can be calculated using procedures defined in Building Code Requirements for Structural Concrete (ACI 318-83 Section 11.7.7) [24].

  Basis: The shear friction capacity defined by ACI 318-71 Section 11.15 does not address members subjected to sustained compression. Later versions of ACI 318 and ACI 349 provide provisions for members subject to permanent net compression when computing shear-friction capacity. The provisions for calculation of shear-friction capacity for members subject to net compression are provided in the following ACI Codes;
  ACI 318-83 and -02 Section 11.7.7, ACI 318-08 and -11 Section 11.6.7, ACI 318-14 Section 22.9.4.5 [24], and ACI 349-85 and -01 Section 11.7.7 [25]. In addition, each of the referenced ACI codes similarly limits the nominal shear stress (or strength), which, in effect, restricts the benefit of permanent net compression. ACI 318-71 Section 11.15.3 limits the maximum nominal shear stress to $0.2f_c{}'$ or 800 psi; more recent code editions such as ACI 318-02 Section 11.7.5 limit the maximum nominal shear strength to $0.2f_c{}'A_c$ or $800A_c$, where A_c is the area of concrete section resisting shear transfer. Both versions of ACI 318-71 and -83 also use the same strength reduction factors, Φ, for shear.

- Supplement 4 – Flexural Cracked Section Properties: The ratio of cracked over uncracked moment of inertia for flexural behavior can be calculated using ACI 318-71 equation 9-4 or it is acceptable to define the cracked moment of inertia as 50% of the gross moment of inertia as discussed below.

  Basis: The flexural stiffness of members with structural cracks or expected to develop structural cracks reduces compared to uncracked section properties. It should be noted that the structural cracks are those due to external loading and not microcracks associated with ASR behavior. The ratio of cracked to uncracked (gross) moment of inertia of 0.5 is consistent with provisions of ACI 318-14 Section 6.6.3.1.2 [17], ASCE 43-05 Table 3-1 [26], and ASCE 4-16 Table 3-2 [27]. Additionally, a review of 24 in. deep sections at Seabrook Station structures with 3 in. concrete cover and reinforcement configurations ranging from No. 8 bars at 12 in. spacing (ratio = 0.0032) to No. 11 bars at 6 in. spacing (ratio = 0.0128) indicates that the ratio of cracked to uncracked gross moment of inertia ranges from 16% to 47%. The first ratio represents minimum allowable flexural reinforcement per ACI 318-71 Section 10.5.1, and the second ratio represents 80% of the maximum allowable flexural reinforcement per ACI 318-71 Section 10.3.2. This review indicates that using a ratio of 50% for reinforced sections in analysis is reasonable or conservative, since flexural demands from ASR tend to decrease as the flexural stiffness decreases. The review also indicates that benefit can be achieved by explicitly calculating and using the cracked section moment of inertia in the analysis. While recent editions of ACI 318 limit the maximum reinforcement ratio via other provisions than contained in ACI 318-71, the effect is similar, and therefore the comparison to the 50% ratio is justified.

- Supplement 5 – Axial and Shear Cracked Section Properties: Axial and shear cracking reduces the corresponding stiffness of a structural member. The effect of cracking on reducing the axial and shear stiffness of structural components may be considered in analysis.

  Basis: As the net tension on a concrete section reaches or exceeds the concrete tensile stress limit, the tensile stiffness of concrete section is reduced gradually to account for possible aggregate interlocking behavior, which is conservative compared to abruptly reducing the concrete tensile strength to zero. Gradual reduction of concrete tension stiffness results in larger net tension on a section that needs to be resisted by the rebars and hence results in conservative evaluation of rebar strength evaluation. The axially cracked section properties can be calculated based on the stress-strain relationship defined in Methodology Document Appendix A which is based on ACI 224.2R92 [28], and Lu and Panagiotou [29]. The shear cracked section properties can be calculated based on the shear retention factor defined in Methodology Document Appendix A which is based on Červenka V. [30].

6. ASR THRESHOLD LIMITS AND MONITORING

6.1 Methodology to Account for Potential Future ASR Expansion

To qualify a structure with margin, the code allowable limits must be larger than the factored design-basis loading plus factored current ASR loading. This margin is used for demands associated with future ASR expansion. The threshold factor is the design margin expressed as the amount which ASR loads can increase beyond values used in the calculations such that the structure or structural component will still meet the allowable limits of the code. Threshold factor is an outcome of the evaluation, not an input to the analysis methodology approach. Calculation of the threshold factor is done by back-calculating the threshold factor such that when the factored original design load is combined with factored ASR load multiplied by the threshold factor, the total demand is equal to or less than the code allowable limits. A unique threshold factor is calculated for each building based on the available margin, and is used to establish threshold limits for structural monitoring parameters. Threshold factor may be revised based on further analysis by using additional inspection and measurement data and/or a more refined structural analysis method without reducing the code inherent margin of safety.

The development of ASR loads based on field measurements of expansion establishes a set of loads that are applicable at the time the measurements are made. The potential for future internal ASR expansion and ASR expansion of the concrete backfill amplified by the threshold limits, as discussed above, shall be considered in the evaluation and design confirmation of Seismic Category I structures. The threshold limit is confirmed when the structural member demands calculated for total factored design loads plus the factored ASR loads amplified by threshold limit remain equivalent to (or slightly less than) the corresponding capacity of the structure.

Specific methods to monitor each structure shall be recommended to identify if there are noticeable changes in the distribution of ASR within a structure from different expansion rates for different regions, changes in boundaries of a region, new regions of ASR, etc. Specific methods to monitor each structure to identify when the selected threshold factor is being approached shall also be recommended. Guidelines for selection of threshold monitoring measurements are as follows:

1. A subset of the measurements listed in Table 9 should be recommended for threshold monitoring.

2. Threshold monitoring measurements should be performed at a frequency of 36, 18, or 6 months for Stages One, Two, and Three structural evaluations, respectively, in accordance with the monitoring intervals specified in Table 7 (consistent with LAR 16-03).

3. The selection of threshold monitoring measurements should be informed by the analysis/evaluation and should track the behaviors/symptoms that are correlated with ASR progression.

4. Threshold monitoring measurements shall be quantifiable, whenever possible. Qualitative monitoring may supplement quantifiable measurements. In such cases, quantitative aspects of the qualitative conditions should be identified for monitoring whenever possible.

5. Threshold monitoring measurements may be recommended in sets or individually. If a set of monitoring measures are used, the measurements within the set should be averaged and compared to a single threshold limit for the entire set. A measurement set should contain only similar types of measurements (e.g., a single measurement set cannot contain both strain measurements and displacement measurements).

6. The method used to perform threshold monitoring measurements shall be capable of detecting the progression of strains and/or deformations between the previous measurements and the threshold measurements. The measurement method should also be repeatable such that incidental variation between repeated measurements is small relative to the margin between the threshold and baseline conditions.

   If threshold monitoring of in-plane strain of a member is recommended, then monitoring using pin-to-pin in-plane expansion measurements is preferred over CI measurements due to its higher precision and repeatability, except for locations where CI measurements are specified for monitoring the in-plane expansion.

7. If CI measurements instead of pin-to-pin expansion measurements are recommended for threshold monitoring, the monitoring should consist of a set with at least two ASR monitoring grids. This reduces the possibility of incidental spalling of a single crack from having a disproportionately large impact on the monitored cracking index relative to the established limit.

In addition to the above quantitative threshold measurements, qualitative threshold measurements can be specified to monitor the ASR growth. Qualitative measurements include observation of new structural cracks, near surface delamination, local crushing of concrete, structural deformation, etc. The locations of the qualitative measurements are generally identified from the analysis performed to qualify the structure. The purpose, type, and specific location of any qualitative measurement shall be clearly defined to enable reliable and repeatable data collection by field inspectors.

6.2 ASR Threshold Monitoring for Stage One Evaluations

Threshold monitoring measurements should be performed at a frequency of 36 months. Since the Stage One analyses are performed using a conservative approach based on several CI and/or pin-to-pin in-plane expansion locations and other structural deformation parameters, there will be a limited number of threshold monitoring quantitative measurements and several qualitative observation parameters. The quantitative measurements shall be compared to the corresponding specified limits from Stage One analysis evaluation. Similarly, the qualitative threshold measurements should be within the specified description and/or limits for these observations. When the observed variables are below the specified limits, the next threshold monitoring shall be performed within the monitoring frequency of 36 months. If a quantitative or qualitative observation variable approaches the corresponding specified limits, then further evaluations or structural modifications may be considered, as described in Section 7.

6.3 ASR Threshold Monitoring for Stage Two Evaluations

Threshold monitoring measurements should be performed at a frequency of 18 months. Quantitative measurements include in-plane expansion measurements and measurement of additional structural deformations. The quantitative threshold variable could be from one location or from an average of several locations with similar behavior. The quantitative measurement or average of several measurements as defined by the monitoring program shall be compared to the corresponding specified limits from Stage Two analysis evaluation. Similarly, the qualitative threshold measurements should be within the specified description and/or limits for these observations. When the observed variables are below the specified limits, then the next threshold monitoring shall be performed within the monitoring frequency of 18 months. If a quantitative or qualitative observation variable approaches the corresponding specified limits, then further evaluations or structural modifications may be considered, as described in Section 7.

6.4 ASR Threshold Monitoring for Stage Three Evaluations

Threshold monitoring measurements should be performed at a frequency of 6 months. Quantitative measurements include CI in-plane expansion measurements, pin-to-pin in-plane expansion measurements, crack width measurements, and measurement of other structural deformation variables. The quantitative threshold variable for each region could be from one location or from an average of several locations with similar behavior. The quantitative and qualitative measurements specified for each building shall be performed within the required frequency of inspection. The quantitative measurement or average of several measurements, as defined by the structural monitoring program, shall be compared to the corresponding specified limits from Stage Three analysis evaluation. Similarly, the qualitative threshold measurements should be within the specified description and/or limits for these observations. When the observed variables are below the specified limits, then the next threshold monitoring shall be performed within the monitoring frequency of 6 months. If a quantitative or qualitative observation variable approaches the corresponding specified limits, then further evaluations or structural modifications may be considered, as described in Section 7. The structure may need to be re-analyzed if new ASR regions are observed during monitoring or a limiting analysis parameter, such as flexural cracking, that was limiting the backfill pressure is observed.

7. EVALUATION FOR APPROACHING THRESHOLD LIMITS

An administrative limit of 97% of the threshold limit is set in addition to reductions of 90%, 95%, and 100% set in LAR 16-03 for Stage One, Two, and Three threshold limits, respectively. The additional 3 percent margin plus the reduction to threshold factors for Stage One and Two analyses provide time to perform additional inspections to confirm that the limits are being approached and to initiate corrective actions. When the quantitative or qualitative threshold monitoring variables reach the administrative limits following the workflow shown in Figure 5, further structural evaluation in accordance with procedures specified in this methodology document shall be performed to re-evaluate the structure or to consider structural modification to alleviate the concern for the approaching variable(s) to the specified limit(s). More frequent ASR threshold monitoring may also be performed. If a structural modification approach is considered, the as-modified structure shall be evaluated using the procedures and acceptance criteria defined in this methodology document to confirm the as-modified structure meets the ASR susceptibility evaluation; and analysis shall be performed to calculate a new threshold factor for the as-modified structure.

7.1 Stage One Evaluation

The Stage One analysis and evaluation approach generally includes significant conservatism. When a threshold measurement reaches the administrative limits set for the Stage One threshold variables, the structure should retain sufficient reserve margin to allow reevaluation using more detailed analysis procedures defined in this methodology document. The reevaluation shall be performed using the more rigorous, higher Stage Two or Three analysis procedures described in this document. However if further evaluation cannot requalify the structure for higher threshold limits, a structural modification concept may be developed to mitigate the risk of exceeding the project acceptance criteria.

7.2 Stage Two Evaluation

The Stage Two analysis and evaluation approach generally includes some conservatism. When a threshold measurement reaches the administrative limits set for the Stage Two threshold variables, then the structure should retain sufficient reserve margin to allow reevaluation and requalification using more detailed analysis procedures defined in this methodology document. The reevaluation shall be performed using another Stage Two analysis with additional field measurements to better define the ASR affected regions used in the previous evaluation or could be a Stage Three analysis. However if further evaluation cannot requalify the structure for higher threshold limits, a structural modification concept shall be developed to mitigate the risk of exceeding the project acceptance criteria.

7.3 Stage Three Evaluation

The Stage Three analysis and evaluation approach generally has considered most of the analysis approaches defined in this document. However, there is usually some conservatism in this evaluation that can be considered if the monitoring variable(s) reach the administrative limits set for the corresponding threshold variable(s). Further field inspections to better define the ASR impact on the structure may be used to reevaluate the building in accordance with the Stage Three procedures defined in this document. However if further evaluation cannot requalify the structure for higher threshold limits, a structural modification concept shall be developed to mitigate the risk of exceeding the project acceptance criteria; analysis shall be performed to calculate a new threshold factor for the as-modified structure.

8. REFERENCES

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9. TABLES

Table 1 – Evaluation Criteria for Suspected ASR Cracking

Visible Features Suggestive or Indicative of ASR Rating	ASR Classification		
	No	**Possible**	**Likely**
Expansion and/or displacement of elements	None	Some	Structure shows symptoms of increase in concrete volume leading to concrete spalling, displacement, and misalignment of elements
Cracking and crack pattern	None	Some cracking–pattern typical of ASR (i.e., map cracking or cracks aligned with major reinforcement or stress)	Extensive map cracking or cracking aligned with major reinforcement or stress
Surface discoloration	None	Slight surface discoloration associated with some cracks	Line or cracks having dark discoloration with an adjacent zone of light-colored concrete
Exudations	None	White exudations around some cracks	Colorless, jelly-like exudations readily identifiable as ASR gel associated with some cracks
Environment	Dry and sheltered	Outdoor exposure but sheltered from wetting	Parts of components frequently exposed to moisture such as rain, groundwater, or water due to natural function of the structure (e.g., hydraulic dam or bridge)
Overall	Based on reviewer experience, judgment, and information from the particular features listed above		

Table 2 – ASR Severity Zones [14]

Zone	CI / CCI Range (mm/m)
I	0.0 to 0.5*
II	0.5 to 1.0
III	1.0 to 2.0
IV	>2.0

* CI / CCI less than 0.1 mm/m can be excluded from Zone I in the Containment Structure evaluation [14]

Table 3 – ASR-Related Strain Loads for Analysis of the Containment Structure [14]

	Strain Load (mm/m) for Stage 1 Screening Evaluation#		Strain Load (mm/m) for Stage 2 Analytical and Stage 3 Detailed Evaluation#	
Zone	Low	High	Low	High
I	0.1	0.6	0.1	0.5
II	0.4	1.3	0.5	1.0
III	0.8	2.5	1.0	2.0
IV	1.5	*	2.0	**

Strains presented as percentage in [14], but have been converted to mm/m in this document for consistency.

* The high strain load for Zone IV is to be 25% greater than the largest observed strain in the zone from CI measurements and/or visual inspection.

** The largest observed strain in the zone from CI measurements may be used.

Table 4 – Load Combinations for Evaluation of Containment Structure (Modified from Table 3.8-1 of UFSAR to Include ASR Loads and Load Factors)

TABLE 3.8-1 CONTAINMENT LOAD COMBINATIONS AND LOAD FACTORS(5)

			LOADING[6]																	
																R_r (DBA Local Effects)				
Design Conditions	Category	Load Combination Number	Dead Load	Live Load	ASR Load	Test Pressure	Accident Pressure	Test Temperature	Normal Temperature	DBA Temperature	Operating Basis Earthquake	Safe Shutdown Earthquake	Wind Load	Tornado	Normal Pipe Reaction	DBA Thermal Pipe Reaction	Reaction of Ruptured High Energy Pipe	Jet Impingement Loads	Impact of Ruptured High Energy Pipe	Pressure Variations	Design Basis Flood
	Loading Notation		D	L	S_a	P_t	P_a	T_t	T_o	T_a[1]	E_o	E_{ss}	W	W_t	R_o	R_a	R_{rr}	R_{rj}	R_{rm}	P_v[2]	F[3]
Service Load	Test	1	1.0	1.0	1.0	1.0	-	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-
	Normal	2	1.0	1.0	1.0	-	-	-	1.0	-	-	-	-	-	1.0	-	-	-	-	1.0	-
	Severe Environmental	3	1.0	1.0	1.0	-	-	-	1.0	-	1.0	-	-	-	1.0	-	-	-	-	1.0	-
Factored Load	Severe Environmental	4	1.0	1.3	1.0	-	-	-	1.0	-	1.5	-	-	-	1.0	-	-	-	-	1.0	-
	Extreme Environmental	5a	1.0	1.0	1.0	-	-	-	1.0	-	-	-	-	1.0[7]	1.0	-	-	-	-	1.0	1.0
		5b	1.0	1.0	1.0	-	-	-	1.0	-	-	1.0	-	1.0[7,8]	1.0	-	-	-	-	1.0	-
	Abnormal	6a	1.0	1.0	1.0	-	1.5[4]	-	-	1.0[4]	-	-	-	-	-	1.0	-	-	-	-	-
		6b	1.0	1.0	1.0	-	1.0[4]	-	-	1.0[4]	-	-	-	-	-	1.25	-	-	-	-	-
	Abnormal/Severe Environmental	7	1.0	1.0	1.0	-	1.25[4]	-	-	1.0[4]	1.25	-	-	-	-	1.0	1.0	1.0	1.0	-	-
	Abnormal/Extreme Environmental	8	1.0	1.0	1.0	-	1.0[4]	-	-	1.0[4]	-	1.0	-	-		1.0	1.0	1.0	1.0	-	-

(1) Includes effect of normal operating thermal loads and accident loads. For all abnormal load conditions, structure should be checked to assure that accident pressure load without thermal load can be resisted by the structure within the specified allowable stresses for this condition.

(2) Negative pressure variations inside the structure shall not be considered simultaneously with outside negative pressure due to tornado loadings.

(3) For this load case, the design basis flood elevation shall be the max. ground water elevation, i.e., El. +20'-0".

(4) Load cases examined for maximum pressure and its coincident liner temperature and maximum liner temperature with its coincident pressure.

(5) All load factors shall be taken as 1.0 for the design of steel liner.

(6) See Subsection 3.8.1.3 for discussion of loadings.

(7) W_t includes missile effects only.

(8) For this load case, loadings from E_{ss} or W_t included individually.

Note: Section references made in the footnotes of this table refer to sections within the UFSAR [12].

Table 5 – Load Combinations for Evaluation of Containment Internal Structures (Modified from Table 3.8-14 of UFSAR to Include ASR Loads and Load Factors)

TABLE 3.8-14 INTERIOR CONTAINMENT STRUCTURES BASIC LOAD COMBINATIONS AND LOAD FACTORS

Design Conditions	Material / Loading Notations		Load Case Number	LOADING[1] Dead Load and Hydrostatic Load	Live Load	ASR Loads	Accident Pressure	Operational Temperature	Accident Temperature	Operating Basis Earthquake	Safe Shutdown Earthquake	Operational Piping Loads	Accident Piping Loads	Jet Force Reaction	Jet Impingement Loads	Missile Impact Loads	Internal Missile Loads	Stress Limit or Design Criteria
				D	L	S_a	P_a	T_o	T_a	E_o	E_{ss}	R_o	R_a	R_{rr}	R_{rj}	R_{rm}	M	
Normal Load	Structural Steel		1S	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	$\leq F_s$ Per AISC
			2S	1.0	1.0	-	-	-	-	1.0	-	-	-	-	-	-	-	
			3S	0.67	0.67	-	-	0.67	-	-	-	0.67	-	-	-	-	-	
			4S	0.67	0.67	-	-	0.67	-	0.67	-	0.67	-	-	-	-	-	
	Concrete		1C	1.4	1.7	2.0	-	-	-	-	-	-	-	-	-	-	-	ACI 318-71
			2C	1.4	1.7	1.3	-	-	-	1.9	-	-	-	-	-	-	-	
			3C	-	-		-	-	-	-	-	-	-	-	-	-	-	
			4C	1.05	1.28	1.0	-	1.28	-	1.43	-	1.28	-	-	-	-	-	
Unusual Load	Structural Steel [3] [4]	Elastic	5S	0.63	0.63	-	-	0.63	-	-	0.63	0.63	-	-	-	-	-	$\leq F_s$ Per AISC
			6S	0.63	0.63	-	0.63	-	0.63	-	-	-	0.63	-	-	-	-	
			7S	0.63	0.63	-	0.63	-	0.63	0.63	-	-	0.63	0.63	0.63	0.63	0.63	
			8S	0.59	0.59	-	0.59	-	0.59	-	0.59	-	0.59	0.59	0.59	0.59	0.59	
	[3] [4]	Plastic	5S	1.1	1.1	-	-	1.1	-	-	1.1	1.1	-	-	-	-	-	AISC, Part II
			6S	1.1	1.1	-	1.7	-	1.1	-	-	-	1.1	-	-	-	-	
			7S	1.1	1.1	-	1.4	-	1.1	1.4	-	-	1.1	1.1	1.1	1.1	1.1	
			8S	1.1	1.1	-	1.1	-	1.1	-	1.1	-	1.1	1.1	1.1	1.1	1.1	
	Concrete [4]		5C	1.0	1.0	1.0	-	1.0	-	-	1.0	1.0	-	-	-	-	-	ACI 318-71
			6C	1.0	1.0	1.0	1.5	-	1.0	-	-	-	1.0	-	-	-	-	
			7C	1.0	1.0	1.0	1.25	-	1.0	1.25	-	-	1.0	1.0	1.0	1.0	1.1	
			8C	1.0	1.0	1.0	1.0	-	1.0	-	1.0	-	1.0	1.0	1.0	1.0	1.0	

Table 6 – Load Combinations for Evaluation of Seismic Category I Structures other Than Containment (Modified from Table 3.8-16 of UFSAR to Include ASR Loads and Load Factors)

TABLE 3.8-16 CATEGORY I, STRUCTURES OTHER THAN REACTOR CONTAINMENT STRUCTURE OR ITS INTERNALS BASIC LOAD COMBINATIONS AND LOAD FACTORS

				Loading(1), (4)																					
				All Structures							Concrete Only			Certain Structures, Where Appropriate											
																		Pipe Break Loads (Rr)							
Design Conditions	Material		Required Strength	Dead Load and Hydrostatic Load	Live Load	Operational Temperature	Operating Basis Earthquake	Safe Shutdown Earthquake	Wind	Tornado Wind	Lateral Earth Pressuress	Earth Pressure due to OBE	Earth Pressure due to SSE	ASR Loads (6)	Operational Piping Loads	Accident Pressure	Accident Piping Loads	Jet Impingement Loads	Missile Impact Loads	Jet Force Reaction	Internal Missile Loads	Accident Temperature	Design basis Flood	Unusual Snow Load	Stress Limit (5) or Design Criteria
	Loading Notations			D	L	T_o	E_o	E_s	W	W_t	E	H_c	H_s	S_a	R_o	P_a	R_a	R_{rj}	R_{rm}	R_{rr}	M	T_a	F	Ls	
Normal Load	Structural Steel		S	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-			≤F_s Per AISC
				1.0	1.0	-	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-			
				1.0	1.0	-	-	-	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-			
				0.67	0.67	0.67	-	-	-	-	-	-	-	-	0.67	-	-	-	-	-	-	-			
				0.67	0.67	0.67	0.67	-	-	-	-	-	-	-	0.67	-	-	-	-	-	-	-			
				0.67	0.67	0.67	-	-	0.67	-	-	-	-	-	0.67	-	-	-	-	-	-	-			
	Concrete		U	1.4	1.7	-	-	-	-	-	1.7	-	-	2.0	-	-	-	-	-	-	-	-			ACI 318-71
				1.4	1.7	-	1.9	-	-	-	1.7	1.9	-	1.3	-	-	-	-	-	-	-	-			
				1.4	1.7	-	-	-	1.7	-	1.7	-	-	1.7	-	-	-	-	-	-	-	-			
				1.05	1.28	1.28	-	-	-	-	1.28	-	-	1.5	1.28	-	-	-	-	-	-	-			
				1.05	1.28	1.28	1.43	-	-	-	1.28	1.43	-	1.0	1.28	-	-	-	-	-	-	-			
				1.05	1.28	1.28	-	-	1.3	-	1.28	-	-	1.28	1.28	-	-	-	-	-	-	-			
				1.2	-	-	-	-	1.7	-	1.7	-		1.7	-	-	-	-	-	-	-	-			
				1.2	-	-	1.9	-	-	-	1.7	1.9		1.3	-	-	-	-	-	-	-	-			
Unusual Load	Structural Steel	Elastic		0.63	0.63	0.63	-	0.63	-	-	-	-	-	-	0.63	-	-	-	-	-	-	-			≤F_s Per AISC
			S	0.63	0.63	0.63	-	-	-	0.63	-	-	-	-	0.63	-	-	-	-	-	-	-			
			(2)	0.63	0.63	-	-	-	-	-	-	-	-	-	-	0.63	0.63	-	-	-	-	0.63			
			(2)	0.63	0.63	-	0.63	-	-	-	-	-	-	-	-	0.63	0.63	0.63	0.63	0.63	0.63	0.63			
				0.59	0.59	-	-	0.59	-	-	-	-	-	-	-	0.59	0.59	0.59	0.29	0.29	0.29	0.59			
		Plastic		1.1	1.1	1.1	-	1.1	--	-	-	-	-	-	1.1	-	-	-	-	-	-	-			AISC, Part II
			Y	1.1	1.1	1.1	-	-	-	1.1	-	-	-	-	1.1	-	-	-	-	-	-	-			
				1.1	1.1	-	-	-	-	-	-	-	-	-	-	1.7	1.1	1.1	1.1	1.1	1.1	1.1			
			(*)	1.1	1.1	-	1.4	-	-	-	-	-	-	-	-	1.4	1.1	-	-	-	-	1.1			
			(*)	1.1	1.1	-	-	1.1	-	-	-	-	-	-	-	1.1	1.1	1.1	1.1	1.1	1.1	1.1			
	Concrete		U	1.0	1.0	1.0	-	1.0	-	-	1.0	-	1.0	1.0	1.0	-	-	-	-	-	-	-			ACI 318-71
				1.0	1.0	1.0	-	-	-	1.0	1.0	-	-	1.0	1.0	-	-	-	-	-	-	-			
				1.0	1.0	-	-	-	-	-	1.0	-	-	1.0	-	1.5	1.0	-	-	-	1.1	1.0			
				1.0	1.0	-	1.25	-	-	-	1.0	1.25	-	1.0	-	1.25	1.0	1.0	1.0	1.0	-	1.0			
				1.0	1.0	-	-	1.0	-	-	1.0	-	1.0	1.0	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0			
				1.0	1.0	1.0	-	-	1.0	-	-	-	-	1.0	1.0	-	-	-	-	-	-	-	(3)	1.0	
				1.0	1.0	1.0	-	-	1.0	-	-	-	-	1.0	1.0	-	-	-	-	-	-	-	1.0	1.0	

(1) In above load combinations, the peak values of P_a, T_a, R_a, R_{rj}, R_{rr}, R_{rm} and M shall be combined (when acting concurrently) unless time history analysis is performed to justify otherwise.
(2) Elastic cases to be checked for overall stability by the plastic load combination cases as indicated by (*).
(3) For design bases flood load case, elevation shall be the effective maximum ground water elevation i.e., El. + 20'-0".
(4) See Subsection 3.8.4.3 for discussion of loadings.

Table 7 – Structure Deformation Monitoring Requirements

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

Table 8 – Acceptance Criteria for Stability against Overturning, Sliding and Flotation

	Factor of Safety		
Load	**Overturning**	**Sliding**	**Flotation**
Service/Normal load combinations	1.5	1.5	-
Factored/Unusual load combinations	1.1	1.1	-
Dead load and design basis flood load	-	-	1.5

Table 9 – Field Observations and Measurements for Susceptibility Evaluations

Label	Description
AC	Cracking suspect of ASR (visual observations) Qualitative visual observations made of cracking that exhibits visual indications of ASR and ASR-related features, using industry guidelines described in References [7] and [9].
NC	Cracking not suspect of ASR (visual observations) Qualitative visual observations made of cracking that does not exhibit indications of ASR. These cracks may be structural (i.e. caused by stresses acting on the structure) or caused by shrinkage or other mechanisms aside from ASR.
SD	Other structural or material distress (visual observations) Qualitative visual observations made of structural distress, such as buckled plates, broken welds, spalled concrete, delaminated concrete, displacement at embedded plates, damage to coatings, and chemical staining.
CI	Cracking index Quantitative measurement of in-plane cracking on a concrete structural component using the cracking index measurement procedure.
IP	In-plane expansion Quantitative measurement of distance between two points installed at the surface of a concrete component using a removable strain gage. In-plane expansion is computed as the change in length-measurement values recorded at different times.
TS	Through-thickness expansion Quantitative measurement in the through-thickness direction of a concrete component using an extensometer device. Through-thickness expansion is computed as the change in through-thickness measurement values recorded at different times.
CW	Individual crack widths/lengths Quantitative measurement of individual crack widths using either a crack card, an optical comparator, or any other instrument of sufficient resolution. Such measurements shall be accompanied by notes, sketches, or photographs that indicate the pattern of the cracks and their length. Also included in this category are tools that quantify the change in crack widths, such as mountable crack gages, extensometers, and invar wires.
SJ	Seismic isolation joints Quantitative measurement of the width of seismic joints that separate two adjacent structures. Also included in this category are qualitative observations of distress in seals covering or filling isolation joints, such as tears, wrinkles, and bubbles.
DM	Structure dimensions Quantitative measurement of a structure's dimensions or the distance between two adjacent structures. Included in this category are measurements of plumbness of walls, levelness of slabs, and bowing/bending of members.
EQ	Equipment/conduit offsets Quantitative measurement or visual observation of building deformation through the misalignment of equipment and/or the deformation of flexible conduit joints.
OT	Other observations/measurements Any other observations or measurements that can be used to characterize and/or quantify ASR conditions or other conditions of distress.

10. FIGURES

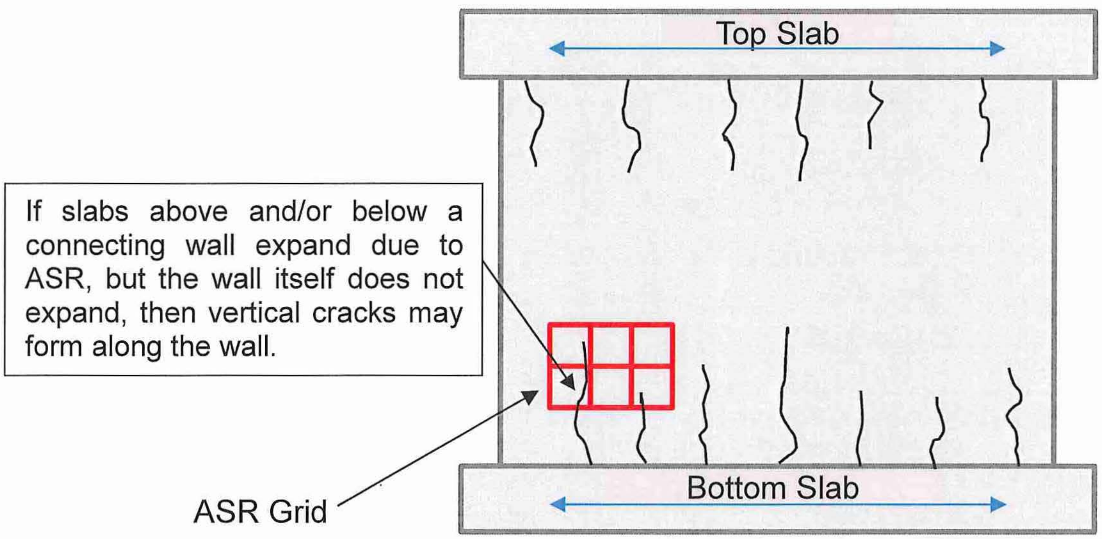

Figure 1. Structural Cracks due to Differential ASR Expansion in the Structure.

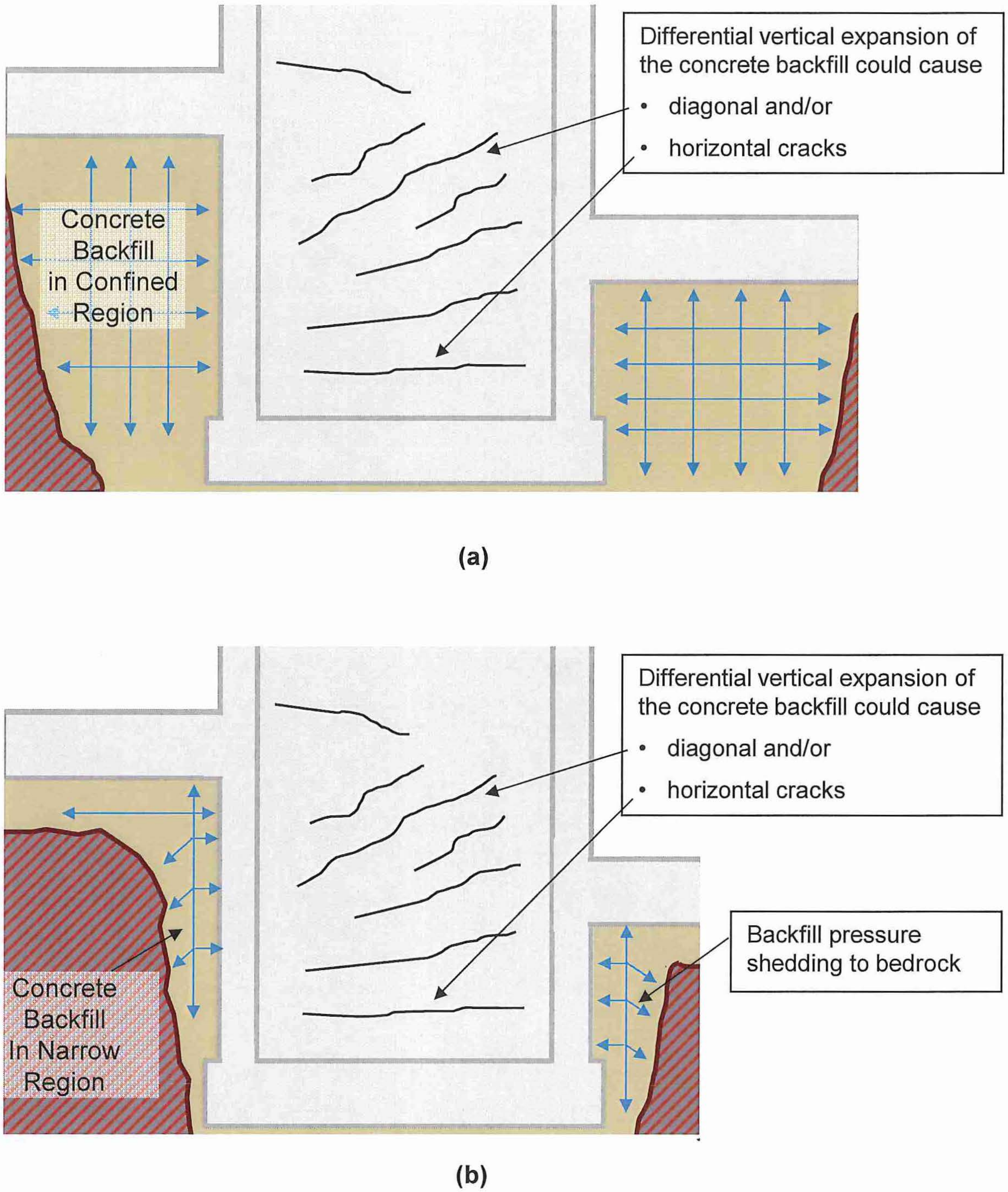

Figure 2. Structural Cracks due to Differential ASR Expansion in the Concrete Backfill.

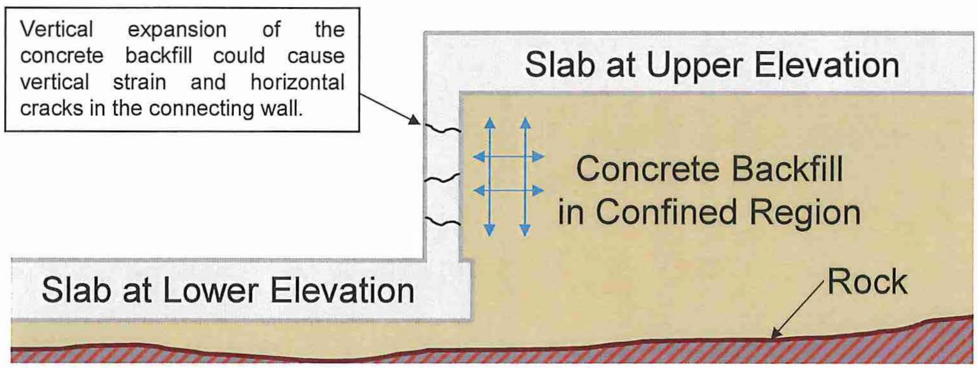

Figure 3. Structural Cracks due to Concrete Backfill Expansion.

Lateral Pressure Exerted by Concrete Fill

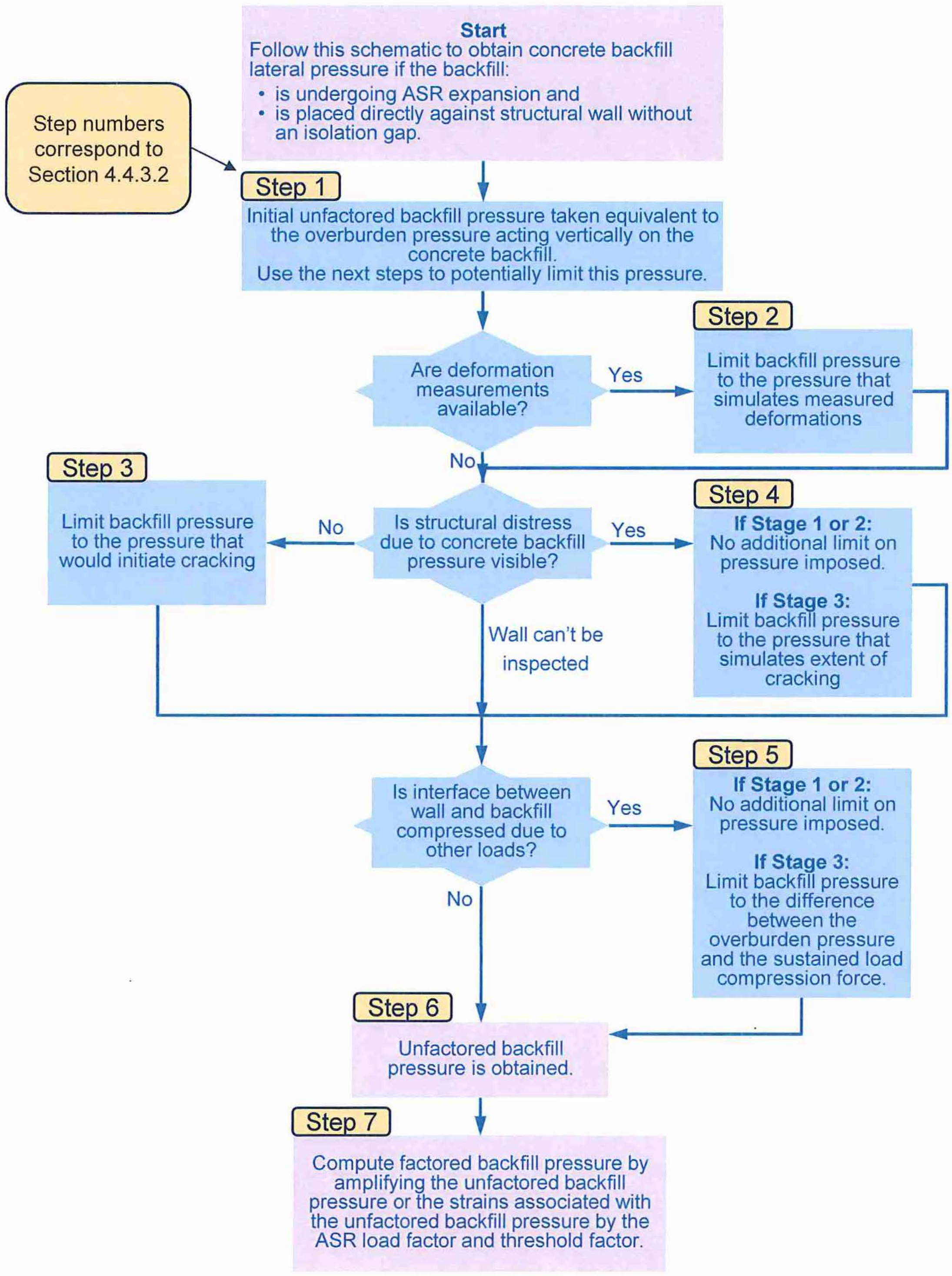

Figure 4. Flow Chart Schematic for Computing Lateral Concrete Backfill Pressure due to ASR.

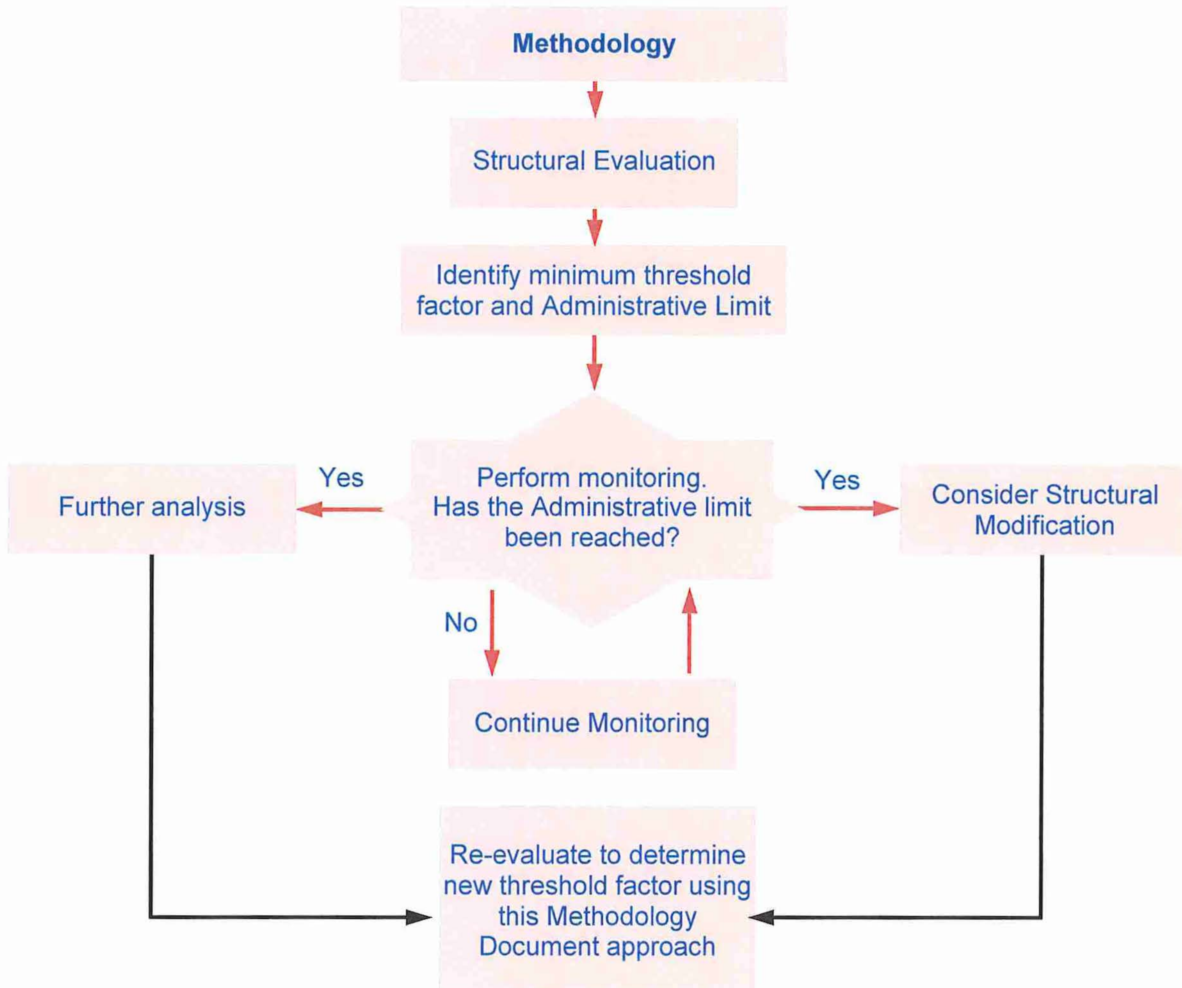

Figure 5. Threshold Monitoring

ATTACHMENT A

APPENDIX A – DETERMINATION OF CRACKED SECTION PROPERTIES

Reinforced concrete cracked section properties shall be determined by following the steps described below:

- **Step One:** Perform the finite element analysis using uncracked section properties.

- **Step Two:** Post-process the results and find the membrane/axial strain, in-plane shear strain, and the out-of-plane moment (M_a) if applicable for each group of elements, i.e. groups of elements that form a region, component, or a member. If the magnitude of moment exceeds the cracking moment and/or axial/membrane strain exceeds the cracking strain, then the elements of that group are cracked.

- **Step Three:** Compare the analytically obtained cracking pattern and axial strains with field observations and measurements. Note that the comparison with field measurements for in-situ condition, i.e. the structure is subjected to unfactored sustained loads.

- **Step Four:** Knowing the axial strain and shear strain either from measurement or analysis, and the bending moment from analysis, calculate the required amount of reduction in flexural, axial and shear rigidities as follows:

  - **Flexural rigidity:** Knowing the cracking moment, M_a, compute the effective moment of inertia using the following ACI 318-79 equation 9-4 [A1]:

    $$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 . I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] . I_{cr}$$

    Where M_a is the demanding moment from analysis, M_{cr} is cracking moment, I_g and I_{cr} are gross and cracked moment of inertias, respectively. This equation implies that as the moment demand (M_a) increases, the effective moment of inertia approaches the cracked moment of inertia.

    To circumvent the need for iteration, the effective moment of inertia can be set equal to $0.5I_g$ but not less than I_{cr} as recommended in ACI 318-14 Section 6.6.3.1.2 [A2].

    To consider the simultaneous effects of axial tension and bending moment, the value of M_{cr} and/or I_{cr} may be computed for a cross-section subjected to tension; however, excluding such an effect is conservative, and hence, is recommended.

    Stiffness reduction coefficient in flexure (ratio of cracked to uncracked flexural rigidity) is:

$$K_f = \frac{(EI)_e}{(EI)} = \frac{I_e}{I_g}$$

- **Axial rigidity:** Knowing the axial/membrane strain, ε, calculate the axial stress, σ, using either the procedure recommended by ACI Committee 224 [A3], or the Steven's equation that accounts for tension stiffening [A4] by using an exponential decay function.

  $$\sigma = f_t\left[(1 - M)e^{-\lambda_t.(\varepsilon-\varepsilon_{cr})} + M\right]$$

  Where

  $$M = 75\ (mm)\frac{\rho}{d_b}$$

  $$\lambda_t = \frac{540}{\sqrt{M}}$$

  $$\varepsilon_{cr} = f_t/E_c$$

  Where ε is the axial strain from finite element analysis or field measurements, σ is the tensile stress, d_b is the diameter of longitudinal rebar in mm, and ρ is the longitudinal reinforcement ratio, f_t is the uniaxial tensile strength of concrete material, and E_c is the elastic modulus of concrete. In above equations, M determines the residual tensile strength and λ_t controls the area under the softening portion of the curve which is related to Mode-I fracture energy. The value of f_t can be computed using the following equation which is within the value range recommended by Nilson et al. [A5]:

  $$f_t = 5\sqrt{f_c'}$$

  The effective concrete modulus is:

  $$E_e = \frac{\sigma}{\varepsilon}$$

  Stiffness reduction coefficient in axial (ratio of cracked over uncracked axial rigidity) is:

  $$K_a = \frac{(EA)_e}{(EA)} = \frac{E_e}{E_c}$$

- **Shear rigidity:** Knowing the shear strain, ε, compute the stiffness reduction coefficient in shear (shear retention factor) for regions with or with expected shear cracks [A6] as follows:

$$K_s = \frac{(GA)_e}{GA} = -\frac{1}{c_2} Ln\left(\frac{1000\varepsilon}{c_1}\right)$$

Where

$$c_1 = 7 + 333(\rho - 0.005)$$

$$c_2 = 10 - 167(\rho - 0.005)$$

ε is the analytically computed shear strain, and ρ is the longitudinal reinforcement ratio.

- **Step Five:** Knowing the stiffness reduction coefficient in flexure (K_f), axial (K_a) and shear (K_s), calculate the cracked section properties for each type of element. Note that the following formulation accommodates any combinations of flexural, axial and shear cracks; for instance if no axial crack was observed (in the field and/or analysis), the equations remain valid by setting $K_a = 1$.

- **Shell elements:** There are five stiffnesses (K_{fx}, K_{fy}, K_{ax}, K_{ay}, K_s defined below) but only four input properties (t_{cr}, $E_{cr.x}$, $E_{cr.y}$, G_{cr} defined below) to an orthotropic shell element (with Poisson's ratio $\nu_{xy} = \nu_{yx} = \nu$); therefore, a membrane element must be introduced. The orthotropic material properties and adjusted shell and membrane thicknesses shall be computed as:

$$t_{cr} = \sqrt{\frac{K_{fx}}{K_{ax}}} \times t$$

$$E_{cr.x} = K_{ax}\sqrt{\frac{K_{ax}}{K_{fx}}} \times E_c$$

$$E_{cr.y} = K_{fy}\left(\sqrt{\frac{K_{ax}}{K_{fx}}}\right)^3 \times E_c$$

$$G_{cr} = K_s \frac{t}{t_{cr}} \times G_c$$

$$E_{cr.y.add} = K_{ay}\frac{t}{t_{cr}} E_c - E_{cr.y}$$

where E_c and G_c are elastic and shear moduli of idealized homogenous concrete material, $E_{cr.x}$ and $E_{cr.y}$ correspond to the orthotropic material elastic moduli of the orthotropic shell element, G_{cr} is the cracked shear modulus of the shell element, t and t_{cr} are the uncracked and adjusted cracked thicknesses of the shell and membrane elements, K_{ax} and K_{ay} are the ratio of cracked to uncracked axial rigidities along two local axis of the shell element, K_{fx} (about y-axis) and K_{fy} (about x-axis) are the ratio of cracked over uncracked flexural rigidities about two local axes of the shell element, and $E_{cr.y.add}$ is the modulus of the additional membrane element along the y-local axis.

Note that the additional membrane element has axial stiffness only in the direction of the larger axial stiffness (parallel to the larger crack direction).

Since the thickness of the shell element is changed, the density must be adjusted to keep the self-weight unaltered. No weight density is needed in the additional membrane element.

$$Dens_{cr} = Dens\frac{t}{t_{cr}}$$

- **Solid elements:** There are six equations and six unknowns, and all equations are independent. The orthotropic material properties shall be calculated as:

$$E_{cr.x} = K_{ax}E_c$$

$$E_{cr.y} = K_{ay}E_c$$

$$E_{cr.z} = K_{az}E_c$$

$$G_{cr.xy} = K_{sxy}G_c$$

$$G_{cr.yz} = K_{syz}G_c$$

$$G_{cr.xz} = K_{sxz}G_c$$

- **Beam elements:** There are four equations and four unknowns, and therefore the orthotropic material properties shall be calculated as:

$$b_{cr} = b\sqrt{\frac{K_{fz}}{K_{ax}}}$$

$$h_{cr} = h\sqrt{\frac{K_{fy}}{K_{ax}}}$$

$$E_{cr.x} = K_{ax}\frac{b \times h}{b_{cr} \times h_{cr}}E_c$$

$$G_{cr} = K_s\frac{b \times h}{b_{cr} \times h_{cr}}G_c$$

$$Dens_{cr} = Dens \frac{b \times h}{b_{cr} \times h_{cr}}$$

where b_{cr} and h_{cr} are adjusted cross-sectional width and height.

- **Step Six:** Modify the finite element model by assigning the calculated cracked section properties to the elements and rerun the analysis. The analysis is considered complete if the overall magnitude of the 1st principal mechanical tensile strain for the majority of cracked elements from two successive steps remain approximately unchanged. If this condition has not been satisfied, go to Step Two and repeat the process. Note that if the analysis stops before obtaining convergence, the results would be conservative, i.e. bending moment, shear and/or tensile force would be greater than the expected values. Therefore, one might use such conservative results for evaluation.

References for Appendix A

[A1] American Concrete Institute, ACI Committee 318, *Building Code Requirements for Reinforced Concrete*, ACI 318-71, Third Printing, 1972.

[A2] ACI 318-14 Building Code Requirements for Structural Concrete and Commentary, 2014

[A3] ACI Committee 224, *Cracking of Concrete Members in Direct Tension*, ACI 224.2R-92, Reapproved 1997.

[A4] Lu, Yuan, and Marios Panagiotou. "Three-dimensional cyclic beam-truss model for nonplanar reinforced concrete walls." Journal of Structural Engineering, 2013, 140(3): 04013071.

[A5] Nilson A. H., Darwin D., and Dolan C. W. Design of concrete structures, 14th Edition, 2010.

[A6] Červenka V., Jendele L., and Červenka J., ATENA Program Documentation, Part 1, Theory. Dec. 2016, pp 29.