March 31, 2017

NRC 2017-0017
10 CFR 50.90

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555

Point Beach Nuclear Plant, Units 1 and 2
Dockets 50-266 and 50-301
Renewed License Nos. DPR-24 and DPR-27

License Amendment Request 278
Risk-Informed Approach to Resolve Construction Truss Design Code Nonconformances

Pursuant to 10 CFR 50.90, NextEra Energy Point Beach, LLC (NextEra) requests an amendment to renewed Facility Operating Licenses DPR-24 and DPR-27 for Point Beach Nuclear Plant, Units 1 and 2, respectively. The proposed change will document a risk-informed resolution strategy to resolve low risk, legacy design code nonconformances associated with construction trusses in the containment buildings of Point Beach, Units 1 and 2.

The proposed License Amendment Request (LAR) is a risk-informed licensing basis change. Accordingly, the proposed change meets the criteria set forth in Regulatory Guide 1.174, "An Approach for Using Probabilistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis," and the generic guidance in Regulatory Guide 1.200, "An Approach for Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities."

Enclosure 1 contains NextEra's evaluation of the proposed change. Enclosure 2 summarizes the regulatory commitments included in this submittal. Enclosure 3 provides a copy of the marked up UFSAR pages for information. Enclosure 4 provides the Probabilistic Risk Assessment evaluation in support of the LAR. Enclosure 5 provides an engineering evaluation summarizing the analyses performed to support the LAR.

NextEra has determined that this LAR does not involve a significant hazard consideration as determined per 10 CFR 50.92. Pursuant to 10 CFR 51.22(b), no environmental impact statement or environmental assessment needs to be prepared in connection with the issuance of this amendment.

NextEra Energy Point Beach, LLC

6610 Nuclear Road, Two Rivers, WI 54241

The proposed LAR has been reviewed by the Point Beach Nuclear Plant Onsite Review Group.

This submittal contains five (5) new regulatory commitments and does not revise any existing commitments. The regulatory commitments are summarized in Enclosure 2.

Given the complexity of the issue and in the absence of precedence, NextEra initially requests approval of this LAR within 24 months. The review schedule may be negotiated with NRC staff. Implementation of the LAR would occur in accordance with the schedule of commitments provided in Enclosure 2.

Pursuant to 10 CFR 50.91(b)(1), a copy of this letter is being forwarded to the State of Wisconsin.

Should you have any questions regarding this submittal, please contact Mr. Bryan Woyak, Licensing Manager, at (920) 755-7599.

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

Executed on March 31, 2017.

Sincerely,

NextEra Energy Point Beach, LLC

Robert Coffey
Site Vice President

Enclosures:

1. NextEra's Evaluation of the Proposed Change
2. Regulatory Commitments
3. Marked Up UFSAR Pages – For Information Only
4. Probabilistic Risk Assessment Evaluation PBN-BFJR-17-019
5. Engineering Evaluation 2017-0008

cc: Administrator, Region III, USNRC
Project Manager, Point Beach Nuclear Plant, USNRC
Resident Inspector, Point Beach Nuclear Plant, USNRC
Public Service Commission Wisconsin

ENCLOSURE 1

NextEra Energy Point Beach, LLC

License Amendment Request (LAR) 278

NextEra's Evaluation of the Proposed Change

CONTENTS

ENCLOSURE 1

NEXTERA'S EVALUATION OF THE PROPOSED CHANGE

1.0 SUMMARY DESCRIPTION

NextEra is requesting an amendment to the current licensing basis for Point Beach Nuclear Plant, Units 1 and 2, to address design code nonconformances associated with the steel construction trusses located in the containment buildings of Units 1 and 2. A risk-informed resolution strategy is proposed to address the nonconformances related to a low risk, legacy condition from original station construction.

The proposed change is acceptance of the final configuration of the construction trusses, including the attached containment spray piping and ventilation ductwork, and the containment liners/walls adjacent to the trusses, using a risk-informed resolution. The risk-based analyses conclude that the final modified configuration of the Unit 1 construction truss, and the current configuration of the Unit 2 construction truss, with implementation of identified safety enhancement modifications to both Units, will not pose a hazard to the safe operation of Point Beach and does not pose a risk to the health and safety of the public.

This License Amendment Request (LAR) proposes a risk-informed licensing basis change. The proposed change meets the criteria set forth in Regulatory Guide 1.174, "An Approach for Using Probabilistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis," Revision 2, and the guidance in Regulatory Guide 1.200, "An Approach for Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities," Revision 2.

2.0 DETAILED DESCRIPTION

The construction trusses in each Unit were originally installed to provide support for the containment dome liner and initial dome concrete pour during original station construction. After the initial concrete pour cured, the truss structures were lowered a few inches away from the containment liner, no longer providing structural support to the dome, and remained in place. The trusses were then used as an attachment point for containment spray piping, ventilation ductwork, post-accident containment ventilation (PACV) piping, and miscellaneous lights and associated conduits. An initial analysis of seismic adequacy was performed by the construction vendor.

The construction trusses were subsequently reanalyzed and walkdowns and reviews of plant photos discovered a discrepancy between the as-built configuration of the trusses and the design drawing that the analysis was based on. Specifically, the lower diagonal bracing framework of the trusses, and the bottom lower diagonal bracing location on the truss, were different than shown on the design drawing. Consequently, these activities and the refinements of the analysis resulted in identifying nonconformances to the design code of record, "AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," April 1963, 6th Edition, for postulated seismic loads. Follow-on inspection of the trusses during initial resolution activities further identified a nonconformance with regard to the available clearance between a limited number of locations on the construction trusses and the containment liner in each Unit.

The following nonconformances are being tracked in the site's Corrective Action Program:

- For postulated design basis event (DBE) seismic loading:
  - the Unit 1 and 2 construction trusses are nonconforming to the design code of record, "AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," April 1963, 6th Edition,
  - the attached containment spray ring header piping and supports are nonconforming to the design codes of record, USAS B31.1 "Power Piping," 1967 Edition, and "AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," April 1963, 6th Edition, respectively, and
  - the containment liner/wall (in localized areas) is currently nonconforming to the design codes of record, ASME Pressure Vessel Code, Section III, and ACI 318-63, "Building Code Requirements for Reinforced Concrete."

- For postulated design basis accident (DBA) thermal loading, which includes thermal expansion (accounting for the insufficient clearance between the trusses and the containment liner at a limited number of locations):
  - the Unit 1 and 2 construction trusses are nonconforming to the design code of record, "AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," April 1963, 6th Edition, and
  - the containment liner/wall (in localized areas) is currently nonconforming to the design codes of record, ASME Pressure Vessel Code, Section III, and ACI 318-63, "Building Code Requirements for Reinforced Concrete."

Detailed analyses have been performed to demonstrate that the current condition is operable but nonconforming to the design code of record. The initial operability analyses were reviewed by Region III staff (Reference 6.3). Operability is assured by analyses concluding structural integrity is maintained and that the trusses remain capable of supporting containment spray piping and ventilation ductwork without affecting the ability of the supported systems to perform their intended design functions. The affected structures/components are passive and there is no adverse effect on accident mitigation strategies. No compensatory measures are necessary to address the current condition.

One modification has been completed on both Units 1 and 2 (reference Section 3.3.1). The bearing box attachment bolt configuration on the truss support brackets was identified as not matching the station drawings. A modification was made to each Unit that returned the bearing box bolted configuration to the as-designed condition.

This LAR describes an additional modification that will be made to the Unit 1 construction trusses to improve clearance between the trusses and the containment liner at specified locations around the containment circumference (reference Section 3.3.2). This modification is necessary to achieve reduced stress levels. Additionally, a modification will be made to one containment spray pipe support in Unit 1 to achieve additional seismic capacity. No new modifications are proposed for Unit 2.

This LAR proposes a risk-informed approach that concludes the final modified configuration of the Unit 1 construction truss, and the current configuration of the Unit 2 construction truss, with implementation of identified safety enhancement modifications to both Units, will not pose a hazard to the safe operation of Point Beach and does not pose a risk to the health and safety of the public. Section 3.3 provides a summary of the pending modifications and also describes the modifications that will be performed to enhance availability of the reactor coolant system feed and bleed capability that will result in additional safety margin. The risk-informed approach supports the limited scope of modifications to resolve the low risk, legacy nonconformances.

This LAR will document the risk-informed resolution of code nonconformances associated with the construction trusses, equipment supported by the trusses, and the containment/containment liner.

The probabilistic risk assessment (PRA) in support of the LAR is provided in Enclosure 4. The PRA analysis addressed seismic and thermal response of the construction trusses. Seismic and thermal fragility analyses were performed in support of this change and are further discussed in Enclosures 4 and 5. The station PRA model does not require modification as a result of this license amendment. No operator actions or time validated actions are impacted by this license basis change. The results of the risk-informed analyses are consistent with the Commission's Safety Goals for public health and safety.

2.1 System/Component Descriptions

2.1.1 Construction Trusses

The construction trusses are comprised of 18 individual truss elements connected and spaced in a circular pattern around the center of each Unit's containment, positioned under the containment dome. Each individual truss element is supported by a bearing block that rests on and is restrained laterally by a bearing housing. The bearing housing is supported by a bracket that is embedded and supported within the containment wall. The bearing housings are bolted to the top flange of the support bracket through slotted holes in the base plate of the bearing housing. The use of slotted holes and graphite plates located between the bearing housing and the support bracket permit the truss to expand and contract radially in response to temperature changes.

During original construction of the pre-stressed concrete containment domes for Units 1 and 2, the construction trusses were installed to provide structural support for the containment dome liner and the initial dome concrete pour. After the initial concrete pour cured, the truss structures were lowered a few inches away from the containment liner, no longer providing structural support to the dome, and remained in place. The trusses were utilized as an attachment point for the containment spray ring headers, the containment air recirculation cooling system (VNCC) ductwork, PACV piping, and miscellaneous lights and associated conduits.

The trusses are constructed from ASTM A36 steel with a combination of welded and bolted connections. The specified standard for the associated truss bolts is ASTM A325, 1964. The design code of record is "AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," April 1963, 6th Edition.

The trusses were not included in the original FSAR seismic classification tables. They were subsequently added to the UFSAR in 2013 as a Seismic Class I structure supporting Class I piping and ductwork.

2.1.2 Containment Liners

The Point Beach containment structures are a right cylinder with a shallow domed roof. The structures have a nominal 3 ft. 6 in. thick concrete cylindrical wall and 3 ft. thick concrete dome with a steel liner. The concrete containments are prestressed and post tensioned. The structures have a 1/4 in. thick welded ASTM A442 steel liner attached to the inside face of the concrete shell to ensure a high degree of leak tightness. The 1/4 in. thick plate is attached to the concrete by an angle grid system stitch welded to the liner plate and embedded in the concrete. The frequent anchoring prevents significant distortion of the liner plate during accident conditions and ensures that the liner maintains its leak tight integrity. The containment structures of Unit 1 and Unit 2 are designed to maintain leakage no greater than 0.2%/24 hours of containment air weight at a design pressure of 60 psig and associated temperature of 286°F.

The steel containment liner is not exposed to the environment or temperature extremes during routine operation. Containment ambient temperature during operation is between 50°F and 120°F.

The containment and containment liner are Seismic Class I structures.

2.1.3 Containment Spray Piping

The containment spray system is credited for containment heat removal following a Loss of Coolant Accident (LOCA) or Main Steam Line Break (MSLB) in containment, iodine and particulate removal from containment following a LOCA, and transfer of sodium hydroxide from the spray additive tank to the containment.

There are two trains of containment spray piping for each Unit. Each train of spray piping and the associated spray nozzle ring headers are attached to the containment construction trusses via welded structural pipe supports and anchors, designed and analyzed for applied seismic loads or thermal conditions.

The containment spray piping is designed to USAS B31.1, “Power Piping,” 1967 Edition.

The containment spray piping is Seismic Class I.

2.1.4 Containment Air Recirculation Cooling Ductwork

The containment air recirculation cooling system is credited for recirculating and cooling the containment air following a LOCA or MSLB inside containment to limit containment temperatures and pressures to less than the containment design limits (286°F and 60 psig, respectively).

The associated containment cooling fans and cooling coils are located at a containment elevation below the construction trusses. A portion of the ventilation ducts are attached to the trusses.

There is no specific design code identified for the ductwork in the original specification. The ductwork and supports were evaluated using the allowable loads specified in the AISC Steel Construction Manual.

2.1.5 Post-Accident Containment Ventilation (PACV) Piping, Lighting and Associated Conduit

PACV piping, containment light fixtures and associated conduit are attached to the construction trusses. The PACV system was originally designed to facilitate use of containment hydrogen recombiners. The PACV system currently has no design basis accident mitigation function. The containment lighting and associated conduits do not have an accident mitigation function.

2.2 Current Licensing Basis Requirements

2.2.1 Technical Specifications

Technical Specification (TS) 3.6.1, "Containment," addresses operability and surveillance requirements for the containment structures. The containment liner is part of the containment structure.

TS 3.6.6, "Containment Spray and Cooling Systems," addresses operability and surveillance requirements for the containment spray and containment cooling systems.

No changes are proposed to the Technical Specifications.

2.2.2 Updated Final Safety Analysis Report (UFSAR)

The general design criteria (GDC) defining the principal criteria and safety objectives for the design of the station are stated in the Point Beach UFSAR. The Point Beach GDC pre-date the GDC subsequently published in 1971.

GDC	POINT BEACH GDC DESCRIPTION
2	PERFORMANCE STANDARDS Those systems and components of reactor facilities which are essential to the prevention or to the mitigation of the consequences of nuclear accidents which could cause undue risk to the health and safety of the public shall be designed, fabricated, and erected to performance standards that enable such systems and components to withstand, without undue risk to the health and safety of the public, the forces that might reasonably be imposed by the occurrence of an extraordinary natural phenomenon such as earthquake, tornado, flooding condition, high wind, or heavy ice. The design bases so established shall reflect: (a) appropriate consideration of the most severe of these natural phenomena that have been officially recorded for the site and the surrounding area and (b) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design.
10	REACTOR CONTAINMENT The containment structure shall be designed (a) to sustain, without undue risk to the health and safety of the public, the initial effects of gross equipment failures, such as a large reactor coolant pipe break, without loss of required integrity, and (b) together with other engineered safety features as may be necessary, to retain for as long as the situation requires, the functional capability of the containment to the extent necessary to avoid undue risk to the health and safety of the public.
41	ENGINEERED SAFETY FEATURES PERFORMANCE CAPABILITY Engineered safety features, such as the emergency core cooling system and the containment heat removal system, shall provide sufficient performance capability to accommodate the failure of any single active component without resulting in undue risk to the health and safety of the public.

49	REACTOR CONTAINMENT DESIGN BASIS The reactor containment structure, including openings and penetrations, and any necessary containment heat removal systems, shall be designed so that the leakage of radioactive materials from the containment structure under conditions of pressure and temperature resulting from the largest credible energy release following a loss-of-coolant accident, including the calculated energy from metal-water or other chemical reactions that could occur as a consequence of failure of any single active component in the emergency core cooling system, will not result in undue risk to the health and safety of the public.
52	CONTAINMENT HEAT REMOVAL SYSTEMS Where an active containment heat removal system is needed under accident conditions to prevent exceeding containment design pressure, this system shall perform its required function, assuming failure of any single active component.

No relief from GDC requirements is requested. No change is required to existing station accident analyses or accident mitigation.

The containment structure, including the containment liner, construction trusses, containment spray system piping, and containment ventilation ductwork are classified as Seismic Class I. Seismic Class I is defined as:

> *"Those structures and components including instruments and controls whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of excessive amounts of radioactivity. Also, those structures and components vital to safe shutdown and isolation of the reactor."*

Components, systems, and structures classified as Class I are designed in accordance with the following criteria:

- Primary steady state stresses, when combined with the seismic stresses resulting from a response spectrum normalized to a maximum ground acceleration of 0.04g in the vertical direction and 0.06g in the horizontal direction simultaneously, are maintained within the allowable stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards, e.g., ASME Boiler and Pressure Vessel Code, USAS B31.1 Code for Pressure Piping, ACI 318 Building Code Requirements for Reinforced Concrete, and AISC Specifications for the Design and Erection of Structural Steel for Buildings.

- Primary steady state stresses when combined with the seismic stress resulting from a response spectrum normalized to a maximum ground acceleration of 0.08g acting in the vertical direction and 0.12g acting in the horizontal direction simultaneously, are limited so that the function of the component, system or structure shall not be impaired as to prevent a safe and orderly shutdown of the plant.

Seismic Class I equipment is analyzed using the license basis Housner ground response spectrum defined for the Point Beach site. These original spectrum response curves were generated by the time history technique of seismic analysis. The sample earthquake utilized was that recorded at Olympia, Washington 45N-120W on April 13, 1949. The originally recorded earthquake was scaled to that of .06g. The curves were generated by applying the recorded earthquake to a single degree of freedom system, for which the values for damping and natural frequency were varied. Some averaging of the curves was provided to smooth out the erratic response of the earthquake's random behavior. At the high frequency end of the curve, the acceleration levels converge to the peak input value at the location inside the building.

Damping values as input to seismic analyses are defined in UFSAR Table A.5-2. The damping values for the pre-stressed concrete containment structure includes the soil-structure interaction damping.

Additional guidance is contained in Point Beach Design Guide DG-C03, "Seismic Design Criteria Guideline," that increases allowable stresses for Safe Shutdown Earthquake loading to 1.5 times normal allowable stresses established in AISC, not to exceed 0.9 times the material yield stress.

2.3 Reason for Proposed Change

This LAR proposes acceptance of the final configuration of the construction trusses, equipment supported by the trusses, and the containment/containment liner using risk-informed resolution.

The PRA analyses conclude acceptance of the final configuration, with implementation of a modification to Unit 1 to improve clearance between the trusses and the containment liner, and the current configuration of Unit 2, and safety enhancement modifications to both Units, results in a calculated change in core damage frequency (ΔCDF) and large early release frequency (ΔLERF) that is within the acceptance guidelines shown in Regulatory Guide 1.174, Figures 4 and 5, and is consistent with the intent of the Commission's Safety Goal Policy Statement.

The construction trusses are located in the containment buildings, with the support brackets located approximately 60 feet above the operating floor elevation. The location of the trusses and the relative distance between support points, combined with limited safe access points and limited personnel/working platforms presents specific risks, such as the rigging and handling of large structural members at extreme elevations that could present an unnecessary risk to the station from a potential dropped object or unnecessary risk to worker safety. The risk-informed resolution limits the scope of modifications to resolve the low risk code nonconformances related to a legacy condition. The PRA analyses indicate minimal risk margin is gained by performance of additional modifications.

2.4 Description of Proposed Change

Acceptance of the final modified configuration of the Unit 1 construction truss and associated equipment, and the current configuration of the Unit 2 construction truss and associated equipment, using a risk-informed approach for resolution.

3.0 TECHNICAL EVALUATION

3.1 Probabilistic Risk Assessment (PRA)

3.1.1 PRA Analysis Summary

A PRA analysis was performed to determine the change in risk associated with seismic and thermal events for the proposed final configuration of the construction trusses and the containment spray piping supported by the trusses and containment liner which can come in contact with the trusses during a seismic or thermal event.

Engineering calculations were performed to support the PRA. The calculations:

- determined the associated strength capacity of the trusses, containment liner/structure, and supported equipment for either applied seismic loading or thermal loading, and
- demonstrated that the construction trusses will retain their structural stability and will not catastrophically fail or result in a seismic II/I interaction (dropped object) as a result of a design basis seismic or thermal event (reference Section 3.2).

The PRA analysis performed a bounding case that conservatively assumes a truss structural failure will always occur when the truss is overstressed and always leads to core damage, i.e., an assumed conditional core damage probability (CCDP) equal to 1.0. This conservative analysis provides an upper limit of the risk metrics that inherently includes the worst credible outcome of all known possible outcomes of a construction truss overstress event and postulated resultant failure, and as such, is bounding both in terms of the potential outcome and the likelihood of outcome. This approach is also consistent with Regulatory Guide 1.174 that states that a PRA should include a full understanding of the impacts of the uncertainties through either a formal quantitative analysis or a simple bounding or sensitivity analysis.

Seismic and thermal fragility analyses were used to calculate the conditional failure probabilities which, when combined with the site specific hazard data, determine core damage frequency (CDF). A modification will be performed to improve the clearance between the construction truss and the containment liner in Unit 1 at specified locations around the circumference of containment for stress/load reduction. This modification is discussed in the PRA analysis (Enclosure 4), Engineering Evaluation (Enclosure 5) and Section 3.3.2.1 below.

A demonstrably conservative analysis was also performed to assess the level of conservatism in the bounding analysis using a limited scope PRA. The demonstrably conservative analysis applied more realistic, yet conservative, assumptions that supported a basis for a CCDP equal to less than 1.0. The conservative assumption that the construction truss will fail if overstressed was retained. Using event trees, conservative failure rates were applied to mitigating systems and an assessment was performed of operator actions. The demonstrably conservative analysis shows there is significant risk margin contained within the bounding case.

In the final modified configuration of Unit 1 with improved clearance between the trusses and the containment liner, and the current configuration of Unit 2, and with implementation of the safety enhancement modifications identified for Units 1 and 2, the change in total risk associated with acceptance of the proposed configuration is within acceptance guidelines shown in Regulatory Guide 1.174, Figures 4 and 5. Consequently, the change is consistent with the intent of the Commission's Safety Goal Policy Statement.

TOTAL RISK	ΔCDF[1]	ΔLERF[1]
Bounding Case w/convolved IPEEE	2.61E-06	5.22E-07
Demonstrably Conservative Case w/convolved IPEEE	1.34E-07	2.68E-08

[1]The change in risk is a comparison to full design compliance using AISC N690-1994.

3.1.2 PRA Quality

The PRA in support of this LAR is subject to quality control as required by Regulatory Guide 1.174. The NextEra PRA process requires the use of qualified individuals, procedures that require calculations to be subjected to independent review and verification, record retention, peer review, and a corrective action program that ensures appropriate actions are taken when errors are discovered. For this activity, the peer review of the PRA analysis was conducted by Jensen Hughes, Inc., and the peer review of the seismic and thermal fragility analyses was conducted by MPR Associates, Inc. The engineering calculations and report in support of the LAR were developed under a vendor Appendix B program and the Engineering Evaluation was created using station procedures and processes. All supporting analyses are subject to 10 CFR 50 Appendix B for quality assurance.

3.1.3 Five Key Principles

The five key principals involved in risk-informed decisionmaking as described in Regulatory Guide 1.174, Revision 2, are addressed herein. This section was prepared with consideration of insights from Regulatory Guide 1.174, Draft Revision 3.

1. ***The proposed change meets the current regulations unless it is explicitly related to a requested exemption (i.e., a specific exemption under 10 CFR 50.12, "Specific Exemptions").***

Detailed analyses have been performed to demonstrate that the current condition is operable but nonconforming to the design codes of record. The specific nonconformances are stated above, in Section 2.0, Detailed Description. The analyses conclude the construction trusses and supported equipment maintain structural stability and the containment liner remains intact. The initial operability analyses were reviewed by Region III staff (reference 6.3).

The applicable station General Design Criteria are identified in Section 2.2.2 above. No relief from GDC requirements is requested. No exemptions from regulations are required.

This LAR proposes a risk-informed licensing basis change related to legacy code nonconformances associated with the construction trusses, equipment supported by the trusses, and the containment liners in Units 1 and 2. In the final modified configuration of Unit 1, with improved clearance between the trusses and the containment liner, and the current configuration of Unit 2, with implementation of safety enhancement modifications to both Units, the PRA analyses conclude the total risk is within the acceptance guidelines shown in Regulatory Guide 1.174, Figures 4 and 5.

This change affects passive components and structures. No changes to the station accident analyses or accident mitigation strategies are required. No revisions to the station Technical Specifications are required.

NRC approval of this LAR will bring the station in compliance with a revised plant licensing basis.

2. ***The proposed change is consistent with a defense-in-depth philosophy.***

The impact of the proposed change was evaluated and determined to be consistent with the defense-in-depth philosophy. The defense-in-depth philosophy in reactor design and operation requires multiple means or barriers to be in place to accomplish safety functions and prevent the release of radioactive material.

The change is assessed against the following defense-in-depth elements:

<u>A Reasonable Balance is Preserved Among Prevention of Core Damage, Prevention of Containment Failure, and Consequence Mitigation:</u>

Seismic and thermal event PRA analyses were performed that identified a small risk associated with the nonconforming conditions, following implementation of a modification to Unit 1 to improve clearance between the construction truss and the containment liner to achieve reduced stress levels and implementation of safety enhancement modifications to both Units. Engineering calculations confirmed that the construction trusses and components supported by the trusses retain their structural capability during a design basis seismic or thermal event without failure and impact on the containment liner does not result in a breach of the liner material. There is no change to station accident response. Consequently, the three fission product barriers are not impacted and this change maintains a reasonable balance among prevention of core damage, prevention of containment failure, and consequence mitigation.

Over-Reliance on Programmatic Activities as Compensatory Measures Associated With the Change in the LB is Avoided:

The construction trusses and equipment supported by the trusses are passive components. No new human intervention is required for routine station operation or accident mitigation.

A modification will be performed to reduce stress levels in the Unit 1 construction truss. Modifications will be performed to enhance protection for the reactor coolant system feed and bleed capability in both Units.

The station structural monitoring program, procedure NP 7.7.9, performs aging management reviews of civil structures and components within the scope of 10 CFR 54, "License Renewal Rule," and 10 CFR 50.65, "Maintenance Rule." Inspection of the trusses is an existing activity for evaluating the structures for Maintenance Rule and License Renewal implementation. The construction truss structural steel members and truss supports are inspected by qualified personnel on a nominal five year frequency.

New seismic and thermal event limits will be established to ensure the construction trusses and equipment supported by the trusses, as well as the containment liners, are inspected and/or analyzed if an event occurs that results in exceeding elastic stress limits (reference Section 3.2). This activity is necessary to ensure the construction trusses, equipment supported by the trusses, and containment liner, as applicable, are evaluated for their ability to withstand a subsequent design basis seismic or thermal event. The new seismic and thermal event limits do not impose administrative actions for frequently occurring events. Therefore, the change does not result in over-reliance on programmatic activities as compensatory measures.

System Redundancy, Independence, and Diversity are Preserved Commensurate with the Expected Frequency, Consequences of Challenges to the System, and Uncertainties (e.g., No Risk Outliers):

Engineering calculations were performed that conclude the construction trusses, equipment supported by the trusses, and the containment and containment liner will continue to perform their designated design functions during a design basis seismic or thermal event. The affected structures/components are passive and there is no change to existing design functions or accident mitigation strategies. Consequently, there is no loss of redundancy for the containment spray or containment air recirculation cooling systems, and there is no loss of the containment fission product barrier. Although failures are postulated in the PRA analysis, the engineering evaluations support the conclusion that the construction trusses and the associated structures/components remain structurally sound and there is no adverse impact or change to any other station structure, system or component (SSC) design function and there is no change to accident mitigation response. Therefore, the proposed change maintains redundancy, independence, and diversity in the affected systems.

Defenses Against Potential Common-Cause Failures are Preserved, and the Potential for the Introduction of New Common-Cause Failure Mechanisms is Assessed:

The current configuration of the Unit 1 and 2 construction trusses and the proposed modification to the Unit 1 construction truss have been evaluated for response to a seismic event. A seismic event has the potential to be a common cause initiator to an overstress condition of the construction trusses in both Units. The engineering calculations verify structural stability during a design basis seismic event. Consequently, there is no adverse impact on the design function of any SSC.

A thermal event would be Unit-specific and would not be a common cause to postulated failure of the trusses. The engineering calculations conclude structural stability is maintained in the current condition, and following the proposed modification to the Unit 1 construction truss. Consequently, there is no common cause related to postulated failure of the systems supported by the trusses.

Station walkdowns performed for issue resolution have not identified any further nonconformances with the as-designed configuration that could present a common cause challenge.

Independence of Barriers is Not Degraded:

The three fission product barriers for each Unit will remain intact as a result of the postulated seismic or thermal events. Although the construction trusses may come in contact with the containment liner as a result of a seismic or thermal event, the engineering calculations conclude adequate structural capability of the liner is maintained during a design basis seismic or thermal event. The concrete containment structure will maintain structural integrity and remains capable of performing the intended design functions. Although failures are postulated in the PRA analysis, the engineering calculations support the conclusion that the reactor coolant system boundary, fuel cladding and containment remain intact during a seismic or thermal event.

The PRA analyses provide documentation that the failure probability of the containment is not significantly changed with acceptance of the final configuration.

Defenses Against Human Errors are Preserved:

Human error is not a contributor to the postulated structural failure. The construction trusses and equipment supported by the trusses are passive components and no new human intervention relative to the structural components or system operations are required during design basis accident response.

This change does not impact any time critical operator actions for accident response.

The Intent of The Plant's Design Criteria is Maintained:

The general design criteria define the principal criteria and safety objectives for the design of the plant. SSCs are classified according to their importance. Class I SSCs are considered essential to the protection of the health and safety of the public. The construction trusses were not originally classified in the station FSAR. The construction trusses were subsequently identified and documented as Class I structures providing support to Class I piping and ductwork. All systems and components designated as Class I are designed so that there is no loss of function in the event of the maximum hypothetical ground acceleration acting in the horizontal and vertical directions simultaneously.

Engineering calculations demonstrate that following completion of the Unit 1 truss and containment spray pipe support modifications, structural integrity is maintained in both Units in the event of a design basis seismic or thermal event, with adequate margin. The trusses, the equipment supported by the trusses and the containment/containment liners remain capable of performing their specified design functions. The subject SSCs are passive structures and components. No new human intervention is required for routine station operation or accident/event mitigation. Therefore, the intent of the station's design criteria is maintained and the station defense-in-depth design is not adversely impacted.

3. ***The proposed change maintains sufficient safety margins.***

This change proposes risk-informed resolution of low risk code nonconformances related to a legacy condition from original station construction. The change affects passive structures. There is no adverse impact to SSCs as a result of this change and all SSCs remain fully capable of performing their designated design basis accident mitigation functions with no change to the method of performing those functions and no need for human intervention.

The change is within the acceptance guidelines shown in Regulatory Guide 1.174, Figures 4 and 5, and is consistent with the intent of the Commission's Safety Goal Policy Statement.

New seismic and thermal event limits will be established to ensure the construction trusses and equipment supported by the trusses, as well as the containment liners, are inspected and/or analyzed, as necessary, if a seismic or thermal event occurs that results in exceeding elastic stress limits (see Section 4.0).

Safety margin is also provided by the ability to implement FLEX, "Diverse and Flexible Coping Strategies" (Reference 6.4). On December 16, 2015, NextEra submitted notification of full compliance with Order EA-12-049 related to mitigation of beyond design basis events (Reference 6.5). In response to the Order, Point Beach implemented beyond design basis mitigation functions that provide defense-in-depth for reactor core cooling and reactor coolant system makeup. Although use of FLEX equipment is not credited in the seismic or thermal PRA analyses, the FLEX Phase 2 equipment is stored and maintained onsite and procedures are available for their use.

The implemented FLEX modifications included, but were not limited to:

- Portable diesel driven charging pumps (PDCP) that can provide borated water from the refueling water storage tank (RWST) or the boric acid storage tanks (BAST) to the Reactor Coolant System (RCS) Loop A or B.

- Portable diesel driven RCS makeup pumps (PDMU) that can provide borated water from the RWST or the BAST to the Residual Heat Removal system connections for RCS injection in MODES 5 or 6 when the refueling cavity is not flooded.

Additionally, modifications related to implementation of NFPA 805, "Performance-Based Standard for Fire Protection for Light Water Reactor Electric Generating Plants," (Reference 6.6) include installation of a 24 hour pneumatic backup supply to the Unit 1 and Unit 2 pressurizer power operated relief valves (PORV) and routing the PORV supply tubing and control cables so they are protected from a postulated dropped object (see Section 4.0). Consequently, the PORV modifications provide protection to increase availability of the reactor coolant system feed and bleed capability.

4. ***When proposed changes result in an increase in CDF or risk, the increases should be small and consistent with the intent of the Commission's Safety Goal Policy Statement.***

Acceptance of the final Unit 1 configuration and the current configuration of Unit 2 results in a calculated change in core damage frequency and large early release frequency that is within the acceptance guidelines shown in Regulatory Guide 1.174, Figures 4 and 5, and is consistent with the intent of the Commission's Safety Goal Policy Statement. The PRA analysis is included as Enclosure 4.

5. ***The impact of the proposed change should be monitored using performance measurement strategies.***

   This change involves passive structures and components.

   The station structural monitoring program, procedure NP 7.7.9, will continue to perform routine visual examinations of the construction trusses and their supports. This is an existing activity for evaluating the structures as part of Maintenance Rule and License Renewal implementation.

   New seismic and thermal event limits will be established to ensure the construction trusses and equipment supported by the trusses, as well as the containment liners, are inspected and/or analyzed if an event occurs that results in exceeding elastic stress limits (see Section 3.2).

3.2 Engineering Evaluations in Support of the LAR

A series of engineering calculations and analyses were performed to support the risk-informed LAR. The supporting calculations and analyses determined the seismic and thermal fragilities in support of the risk assessment and demonstrated that structural integrity, i.e., the ability to support carried loads and not interfere with supported equipment functions, was maintained during a design basis seismic or thermal event. The calculations conclude adequate margin exists for structural stability by using alternate evaluation methods and acceptance criteria as the structures/components evaluated do not meet current design requirements.

A high level summary of the calculations is provided below. A more detailed summary, which includes the methodology for analyzing the design basis seismic event, is included as Enclosure 5.

Seismic Evaluations to Support Risk-Informed Resolution

Evaluations were performed of the construction truss configuration to determine structural stability as a result of a design basis seismic event. With Unit 1 in a modified configuration with improved clearance between the truss structure and the containment liner, and Unit 2 in its current configuration, the trusses still experience nonconformance with the current licensing basis due to exceeding design allowable stresses and due to making contact with the containment liner. Consequently, alternate evaluation methods and acceptance criteria were employed to verify structural stability and containment integrity and support the conclusion that the nonconformances are of low risk.

The construction trusses were evaluated for the following design basis loads, including vertical and horizontal loads, as appropriate:

- Dead load due to self-weight of the construction trusses and attached components (containment spray lines, PACV piping, lights, and containment ventilation ductwork)
- Seismic inertia loads due to dead load
- Seismic loads from attached components

Site-specific ground motion response spectra (GMRS) were utilized to analyze the seismic response of the trusses at the 125 ft elevation of containment, the approximate elevation of the truss support brackets. The GMRS was developed to support the reevaluated seismic hazard for the station in response to the March 12, 2012, request for information pursuant to 10 CFR 50.54(f) (Reference 6.7). The response spectra was developed using the guidance provided in ASCE/SEI 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities," and NUREG-0800, "Standard Review Plan," for the soil structure interaction (SSI) analysis. Earthquake seed motions were developed using NUREG/CR-6728, "Technical Basis for Revisions of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines." Damping values are per ASCE 43-05, with limitations per NUREG/CR-6926. Vertical response spectra (for the truss and supported systems) were developed using the V/H ratios identified in Appendix A of EPRI High Frequency Program Technical Report 3002004396. For complete discussion of analysis methods and acceptance criteria that were employed refer to Enclosure 5, Section 5.

The results of the analysis show that, while a majority of members and connections were within design allowable stresses, stresses in select chord segments of the trusses in each Unit exceed material yield strength. The resulting strains were shown to remain within defined strain limits. The criteria for acceptability of the developed strains are documented in Enclosure 5, Section 5. Consequently, the results conclude that no structural failure occurs during a design basis seismic event and the truss maintains structural integrity.

Additionally, the truss deflection from applied seismic loads could result in the trusses making contact with the containment liner at the upper chord flange at multiple locations around the containment circumference. Supporting analyses demonstrate that the applied load to the containment liner/structure remained within defined allowable limits, and the containment and containment liner will not experience structural failure or be breached as a result of this contact. The analysis used the actual compressive strength of the containment concrete based on original 90-day test data, adjusted to account for age-hardening beyond the specified 28-day strength. The analysis shows that structural integrity of the liner is maintained during a design basis seismic event and the containment liners will perform their design basis function.

The containment spray piping and pipe supports were determined to be within code limits, except for one support in Unit 1, SI-301R-1-H202, which requires modification as discussed in Section 3.3.2.1 below. Analyses for the VNCC ductwork concluded that stresses remained within design allowable limits, and the PACV piping was demonstrated to meet specified acceptance criteria.

Seismic analyses were also performed to identify the limiting seismic event that will maintain stress in the trusses and adjacent structures within defined allowable/elastic limits. To address continued operation after experiencing this limiting event, and to ensure the affected SSCs remain capable of withstanding a subsequent design basis seismic event, NextEra will implement new seismic operating limits applicable to the Unit 1 and Unit 2 construction trusses and attached components for any seismic event that could result in exceeding elastic stress limits. The applicable bounding limits are:

Horizontal:	0.053g Peak Ground Acceleration
Vertical:	0.045g Peak Ground Acceleration

The existing seismic monitors will be used to detect the new operating seismic limits. The monitors are located in the following locations:

- Unit 1 Façade, 6.5 ft elevation
- Control Building, 8 ft elevation
- Primary Auxiliary Building, 26 ft elevation
- 13.8 kV Building, 26.5 ft elevation

Site procedures will be revised to initiate actions to commence a controlled dual Unit backdown to hot shutdown upon reaching the specified peak ground acceleration limits. The procedure will specify that the Units may be reduced in power individually in series, rather than concurrently, to ensure station staff have the proper resources and focus on each Unit for maximum safety and operational excellence during this off-normal operation. The procedure will also initiate inspection and/or evaluation actions for the construction trusses, equipment supported by the trusses, and the containment liners.

Thermal Evaluations to Support Risk-Informed Resolution

Thermal event analyses were performed to determine structural stability during an event resulting in reaching the design basis containment temperature limit. The limiting event was determined to be a MSLB inside containment.

The construction trusses were evaluated for thermal loads due to differential thermal displacements between the trusses and the containment spray lines and between the trusses and the containment liner.

A modification will be performed to Unit 1 to improve clearance between the construction truss and the containment liner at specified locations around the circumference of the containment for stress/load reduction (reference Section 3.3.2.1). With implementation of this modification to Unit 1, and the current configuration of Unit 2, the thermal event calculations conclude that the containment spray piping and pipe supports remain within design allowable limits. The calculations conclude that the majority of construction truss members meet design allowable limits, but at select locations the stresses exceeded design allowable values, but remain within allowable strain limits. Consequently, the results revealed that no structural failure will occur as a result of a design basis thermal event and the truss maintains structural integrity.

The thermal growth may result in contact with the containment liner at the upper chord flange at multiple locations around the containment circumference in both Unit 1 and Unit 2 as a result of a thermal event, e.g., MSLB. Supporting analyses demonstrate that the applied load to the containment liner/structure remained within defined allowable limits, and the containment and containment liner will not experience structural failure or be breached as a result of this contact. The analysis used the actual compressive strength of the containment concrete based on original 90-day test data, adjusted to account for age-hardening beyond the specified 28-day strength. The analysis shows that structural integrity of the liner is maintained during a design basis thermal event and the containment liners will perform their design basis function.

The containment spray piping and pipe supports were determined to be within code limits for the applied thermal loading. Analyses for the VNCC ductwork concluded that stresses remained within design allowable limits, and the PACV piping was demonstrated to meet specified acceptance criteria.

Thermal analyses were also performed to identify the limiting thermal event that will maintain stress in the trusses and adjacent structures within defined allowable/elastic limits. Therefore, to address continued operation after this limiting event, and to ensure the affected SSCs remain capable of withstanding a subsequent design basis thermal event, NextEra will implement new thermal operating limits applicable to the Unit 1 and 2 construction trusses and attached components for any thermal excursion or occurrence resulting in exceeding elastic stress limits. The proposed new operational containment temperature limits are:

Unit 1: 227°F containment atmospheric temperature
Unit 2: 236°F containment atmospheric temperature

The station implementation of these new limits will include taking the affected Unit offline and performing an inspection and/or evaluation, as necessary, to confirm that the construction truss' future stability during a postulated design basis accident was not compromised by the thermal occurrence.

Detection of the thermal occurrence and initiation of actions to address the new limiting values will be addressed in site procedures. The site procedures will be revised to initiate inspection and/or evaluation actions prior to Unit startup for the affected Unit's construction truss, equipment supported by the truss (as necessary), and the containment liner. No new instrumentation is required to support this change. Existing station instrumentation will be utilized for containment temperature identification. The existing containment temperature indicators provide remote Control Room indication and are capable of identifying noncompliance with the new thermal event limits.

3.3 Modifications

3.3.1 Completed Modifications

During the Unit 2 refueling outage in March 2014, the Unit 2 construction truss bearing box bolts were found not centered in slotted holes as analyzed. A modification was performed to center the bolts at the required locations to return the truss structure to the as-designed condition.

In April 2014, Unit 1 was electively taken off-line and inspected. A similar nonconformance was identified with certain bearing box bolts. The condition was also corrected by modification to center the bolts at the required locations to return the truss structure to the as-designed condition.

3.3.2 Pending Modifications

3.3.2.1 Required Modifications

<u>(a) Unit 1 Spatial Clearance</u>

Upon approval of this LAR, a modification will be made to the Unit 1 construction truss to improve clearance between the truss and the containment liner to achieve reduced stress levels. The modification includes a small amount of material removal at truss upper chord structural tee flanges. The modification will be performed at six specified locations around the circumference of containment. The resulting thermal fragility for the modified Unit 1 condition will be bounded by the Unit 2 thermal risk.

The spatial clearance modification requires approval of this LAR prior to implementation as the small amount of material removal modifies the sectional properties of the construction truss, which impacts the available seismic margin.

No clearance modifications are required for Unit 2.

<u>(b) Unit 1 Containment Spray Pipe Support</u>

As identified in the Engineering Evaluation (Enclosure 5), one containment spray pipe support in Unit 1, SI-301R-1-H202, will be modified to achieve additional seismic capacity. The modification will increase the size of the support's U-bolt diameter.

No containment spray pipe support modifications are required for Unit 2.

3.3.2.2 Modifications to Enhance Safety Margin

Additional protection for the reactor coolant system feed and bleed capability is provided by the following modifications that will be performed as part of NFPA 805 implementation (Reference 6.6). Consequently, these modifications result in additional safety margin.

(a) Unit 1 and Unit 2 Pressurizer PORV Backup Nitrogen Supply

This modification will install a 24 hour pneumatic backup supply (nitrogen) to the PORVs. The modification will also route the nitrogen supply lines so that all tubing outside the pressurizer cubicle is located below the containment operating floor, and thus protected from a postulated falling object.

(b) Reroute Unit 1 and Unit 2 Pressurizer PORV Control Cables

This modification will reroute the pressurizer PORV control cables to prevent them from being exposed to a postulated falling object.

3.4 Technical Evaluation Summary

A PRA analysis was performed to determine the change in risk associated with seismic and thermal events for the proposed final configuration of the construction trusses and the containment spray piping supported by the trusses and containment liner which can come in contact with the trusses during a seismic or thermal event.

Engineering calculations were performed in support of the risk assessment and utilized alternate evaluation methods and acceptance criteria as the structures/components evaluated do not meet current design basis requirements. The alternate evaluation methods and acceptance criteria are not proposed as part of the license basis revision.

Seismic and thermal fragility analyses were created to support the PRA analysis. A modification will be performed to Unit 1 to improve clearances between the construction truss and the containment liner at specified locations around the circumference of the containment for stress/load reduction. One containment pipe support in Unit 1 will be modified to increase its seismic capacity. Modifications will be made to Unit 1 and Unit 2 to provide a 24 hour backup pneumatic supply to the PORVs that will include protecting the supply lines and PORV control cables from a postulated falling object. The PORV modifications will result in additional safety margin by protecting and increasing availability of the reactor coolant system feed and bleed capability.

The engineering analyses support the determination of low risk by concluding the construction trusses, equipment supported by the trusses, and the containment liners maintain structural integrity during a design basis seismic or thermal event. Consequently, there are no changes to any existing design functions and there is no change to station accident response credited in the safety analyses.

New seismic and thermal event limits will be implemented to ensure the construction trusses and associated equipment are inspected and/or analyzed for any seismic or thermal event exceeding elastic stress limits to determine their capability to withstand a subsequent design basis event prior to Unit restart.

The PRA analysis concludes that the total risk associated with the final configuration of the modified Unit 1 construction truss, the current configuration of the Unit 2 construction truss, and with implementation of the identified safety enhancement modifications in Units 1 and 2, is within acceptance guidelines shown in Regulatory Guide 1.174, Figures 4 and 5. Consequently, the change is consistent with the intent of the Commission's Safety Goal Policy Statement.

4.0 REGULATORY EVALUATION

This LAR proposes acceptance of the final configuration of the construction trusses, including the attached containment spray piping and ventilation ductwork, and the containment liners/walls adjacent to the trusses, using a risk-informed resolution. The effects of the change (ΔCDF and ΔLERF) are within the acceptance guidelines shown in Figures 4 and 5 of Regulatory Guide 1.174 and are consistent with the intent of the Commission's Safety Goal Policy Statement.

This LAR will result in a revision to the UFSAR for Point Beach, Units 1 and 2.

This change does not impact the station Technical Specifications.

4.1 Applicable Regulatory Requirements/Criteria

The applicable General Design Criterion and license basis design requirements are addressed in Section 2.2, above. No relief from GDC requirements is requested.

The proposed change meets the criteria set forth in Regulatory Guide 1.174, "An Approach for Using Probabilistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis," and the generic guidance in Regulatory Guide 1.200, "An Approach for Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities."

4.2 No Significant Hazards Evaluation

This LAR proposes a risk-informed approach, in conjunction with modification to the Unit 1 construction truss to improve clearance between the truss and the containment liner, to resolve low risk code nonconformances associated with a legacy condition from original station construction. Modifications associated with implementation of NFPA 805 are planned that will provide protection of the reactor coolant system feed and bleed capability and result in additional safety margin.

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

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

   Response: No

   The probability of an accident previously evaluated is not changed. The containment structures and the containment spray piping and ventilation ducts attached to the construction trusses are accident mitigation equipment. They are not accident initiators.

The acceptance of the final configuration of Point Beach Units 1 and 2 results in a change in core damage frequency and large early release frequency that is within acceptance guidelines and does not involve a significant reduction in the margin of safety. Although failures are postulated in the PRA analysis, the engineering calculations in support of the LAR conclude that the construction trusses and the associated structures/components remain structurally sound in the event of a design basis seismic or thermal event and there is no adverse impact or change to any station SSC's design function and there is no change to accident mitigation response.

This change has no impact on station fire risk caused by a seismic event.

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

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

   Response: No

   The proposed change does not install any new or different type of equipment in the plant. The proposed change does not create any new failure modes for existing equipment or any new limiting single failures. Engineering calculations conclude the construction trusses, equipment supported by the trusses, and containment liners remain capable of withstanding design basis seismic and thermal events and remain capable of performing their designated design functions. Additionally, the proposed change does not involve a change in the methods governing normal plant operation, and all safety functions will continue to perform as previously assumed in the accident analyses. Thus, the proposed change does not adversely affect the design function or operation of any structures, systems and components important to safety.

   There are no new accidents identified associated with acceptance of the final modified configuration of Unit 1 and the current configuration of Unit 2.

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

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

   Response: No

   The effects of the change, ΔCDF and ΔLERF, are within the acceptance guidelines shown in Figures 4 and 5 of Regulatory Guide 1.174. Consequently, the change does not result in a significant reduction in the margin of safety.

The containment structures and liners, construction trusses, and equipment supported by the trusses remain fully capable of performing their specified design functions as concluded by supporting engineering calculations.

Modifications associated with implementation of NFPA 805 are planned that will provide protection of the reactor coolant system feed and bleed capability and result in additional safety margin.

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

The station will implement new seismic and thermal event limits to ensure the construction trusses and associated equipment are inspected and/or analyzed for any event exceeding elastic stress limits to determine their capability to withstand a subsequent design basis event prior to Unit restart.

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

4.3 Conclusion

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

5.0 ENVIRONMENTAL ASSESSMENT

This LAR proposes risk-informed resolution of low risk nonconformances associated with the original design of the containment construction trusses, equipment supported by the trusses and containment liners.

This change does not affect the specified design functions of the impacted structures, systems and components. No new accidents are initiated. The change does not involve a significant reduction in a margin of safety.

NextEra has determined that this proposed change does not involve (i) a significant hazards consideration, (ii) a significant change in the types or significant increase in the amounts of any effluent that may be released offsite, or (iii) result in a significant increase in individual or cumulative occupational radiation exposure. Accordingly, the proposed change meets the eligibility criterion for categorical exclusion set forth in 10 CFR

51.22(c)(9). Therefore, pursuant to 10 CFR 51.22(b), no environmental impact statement or environmental assessment needs be prepared in connection with the proposed change.

6.0 REFERENCES

6.1 Regulatory Guide 1.174, "An Approach for Using Probabilistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis," Revision 2

6.2 Regulatory Guide 1.200, "An Approach for Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities," Revision 2

6.3 NRC Letter, "Point Beach Nuclear Plant, Units 1 and 2 NRC Integrated Inspection Report 05000266/2014004; 05000301/2014004; and 07200005/2014001," dated October 30, 2014

6.4 NEI 16-08, "Guidance for Optimizing the Use of Portable Equipment," Revision 0

6.5 NextEra Energy Point Beach to US NRC, "NextEra Energy Point Beach, LLC's Notification of Full Compliance with Order EA-12-049 Modifying Licenses with Regard to Requirements for Mitigation Strategies for Beyond-Design-Basis External Events and Submittal of Final Integrated Plan," dated December 16, 2015 (ML15350A085)

6.6 NextEra Energy Point Beach to US NRC, "License Amendment Request 271, Transition to 10 CFR 50.48(c) – NFPA 805, Performance-Based Standard for Fire Protection for Light Water Reactor Electric Generating Plants," dated June 26, 2013

6.7 NextEra Energy Point Beach to US NRC, "NextEra Energy Point Beach, LLC Seismic Hazard and Screening Report (CEUS Sites), Response [to] NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," dated March 31, 2014

ENCLOSURE 2

NextEra Energy Point Beach, LLC

License Amendment Request (LAR) 278

Regulatory Commitments

(2 pages follow)

ENCLOSURE 2

REGULATORY COMMITMENTS

The following table identifies the regulatory commitments in this document. Any other statements in this submittal represent intended or planned actions, are provided for information purposes, and are not considered to be regulatory commitments.

COMMITMENT	TYPE		SCHEDULED COMPLETION DATE (if applicable)
	ONE-TIME	CONTINUING COMPLIANCE	
NextEra will implement a modification to Unit 1 to improve clearance between the construction truss and the containment liner at six specified locations around the containment circumference.	X		The modification will be implemented during the refueling outage that falls within the established outage milestones. The Unit 1 modification will occur during the refueling outage that occurs no less than 12 months following license amendment issuance.
Unit 1 containment spray pipe support SI-301R-1-H202 will be modified to achieve additional seismic capacity. The modification will increase the size of the support's U-bolt diameter.	X		This modification will be implemented no later than the refueling outage that the Unit 1 clearance modification is performed.
Modifications will be performed to install a 24 hour backup pneumatic supply to the PORVs and route the supply tubing and electrical cables for the PORVs such that they are protected from a postulated dropped object.	X		Implementation is associated with NFPA 805 requirements. These modifications are required to be implemented no later than prior to startup from the second refueling outage (for each unit) after receipt of the NFPA 805 license amendment which is dated September 8, 2016 (References 1, 2).

COMMITMENT	TYPE		SCHEDULED COMPLETION DATE (if applicable)
	ONE-TIME	CONTINUING COMPLIANCE	
NextEra will implement new seismic operating limits applicable to both Units that maintain stresses in the construction trusses within elastic stress limits. Site procedures will be revised to initiate actions to commence a controlled dual Unit backdown to hot shutdown upon reaching the specified peak ground acceleration limits. The procedure will specify that the Units may be reduced in power individually in series, rather than concurrently, to ensure station staff have the proper resources and focus on each Unit for maximum safety and operational excellence during this off-normal operation. The procedure will initiate inspection and/or evaluation actions for the construction trusses, equipment supported by the trusses, and the containment liners, as necessary.		X	Concurrent with implementation of the modification to Unit 1 that will improve clearance between the construction truss and the containment liner.
NextEra will implement new thermal operating limits applicable to both Units that maintain stresses in the construction trusses from any thermal excursion or occurrence within elastic stress limits. Detection of the thermal occurrence and initiation of actions will be addressed in station procedures and will initiate inspection and evaluation actions prior to Unit startup for the affected Unit's construction truss and equipment supported by the truss, as well as the containment liner.		X	Concurrent with implementation of the modification to Unit 1 that will improve clearance between the construction truss and the containment liner.

REFERENCES:

1. NextEra Energy Point Beach to US NRC, "License Amendment Request 271, Transition to 10 CFR 50.48(c) – NFPA 805, Performance-Based Standard for Fire Protection for Light Water Reactor Electric Generating Plants," dated June 26, 2013

2. US NRC to NextEra Energy Point Beach, "Point Beach Nuclear Plant, Units 1 and 2 – Issuance of Amendments Regarding Transition to a Risk-Informed, Performance-Based Fire Protection Program in Accordance with 10 CFR 50.48(c) (CAC Nos. MF2372 and MF2373)," dated September 8, 2016

ENCLOSURE 3

NextEra Energy Point Beach, LLC
Point Beach Nuclear Plant Unit 1 and 2

License Amendment Request (LAR) 278

Marked Up UFSAR Pages – For Information Only

(5 pages follow)

5.6 CONSTRUCTION

5.6.1 CONSTRUCTION METHODS

5.6.1.1 APPLICABLE CODES

The following codes of practice are used to establish standards of construction procedure:

ACI 301	-	Specification for Structural Concrete for Buildings (Proposed)
ACI 306	-	Recommended Practice for Cold Weather Concreting
ACI 318	-	Building Code Requirements for Reinforced Concrete
ACI 347	-	Recommended Practice for Concrete Formwork
ACI 605	-	Recommended Practice for Hot Weather Concreting
ACI 613	-	Recommended Practice for Selecting Proportions for Concrete
ACI 614	-	Recommended Practice for Measuring, Mixing and Placing Concrete
ACI 315	-	Manual of Standard Practice for Detailing Reinforced Concrete Structures
ASME	-	Boiler and Pressure Vessel Code, Sections III, VIII and IX
AISC	-	Steel Construction Manual
PCI	-	Inspection Manual

5.6.1.2 CONCRETE

Cast-in-place concrete was used to construct the containment shell. The base slab construction was performed utilizing large block pours. After the completion of the base slab steel liner erection and testing, an additional 18 in. thick concrete slab was placed to provide protection for the floor liner.

The concrete placement in the walls was done in 10 ft. high lifts with vertical joints at the radial center line of each of six buttresses. Cantilevered jump forms on the exterior face and the interior steel wall liner served as the forms for the wall concrete.

The dome liner plate, temporarily supported by 18 radial steel trusses and purlins, served as an inner form for the initial 8 in. thick pour in the dome. The weight of the subsequent pour was supported in turn by the initial 8 in. pour. The trusses were lowered away from the liner plate after the initial 8 in. of concrete reached design strength, but prior to the placing of the balance of the dome concrete.

The horizontal and the vertical construction joints were prepared by dry sandblasting followed by cleaning and wetting. Horizontal surfaces were covered with approximately 1/4 in. thick mortar of the same cement-sand ratio as used in the concrete immediately before concrete placing.

5.6.1.3 REINFORCING STEEL

Prior to placing, visual inspection of the shop fabricated reinforcing steel was performed to ascertain dimensional conformance with design specifications and the drawings. This was followed by a check "in place" performed by the placing inspector to assure the dimensional and location conformance.

The trusses are used as a support for containment spray piping, containment air recirculation cooling system (VNCC) ductwork, post-accident containment ventilation (PACV) piping, and miscellaneous lights and associated conduits. See Section A.5.10 for resolution of design code nonconformances related to the trusses and associated components/structures.

A.5.9 SEISMIC ANALYSIS OF THE DIESEL GENERATOR BUILDING (DGB)

The mathematical model of the DGB consisted of several stick elements representing the reinforced concrete shear walls with nodes at each floor level. Each of these nodes was connected by rigid links, representing the rigid diaphragm action of the floor slab. The soil-structure interaction was accounted for by using six soil springs (three translations and three rotations in a Cartesian system), attached to the rigid foundation mat. The Housner horizontal design spectra with a peak ground acceleration of 0.06g for an operating basis earthquake and 0.12g for a safe shutdown earthquake were used as ground input motions. The vertical component of ground acceleration was 2/3 of the magnitude of the horizontal component. The responses (deflections, moments, shears, etc.) of the building were obtained through the response spectrum method using one set of soil spring values.

Insert new A.5.10

Response spectra curves for equipment located in the DGB were obtained through time history analysis. The analysis started with the design earthquake time histories input at the bottom of the mathematic model of the DGB. The time histories for the three directions of motion (two horizontal and one vertical), at each floor were then obtained as a result of the analysis. By applying these floor time histories to a single-degree-of-freedom oscillator, response spectra curves were obtained for each of the floors of the DGB. (Reference 16 and Reference 17)

A.5.11 ~~A.5.10~~ REFERENCES

1. NRC Safety Evaluation dated September 30, 1983, Amendment No. 75 to Facility Operating License No. DPR-24.

2. WE Letter to NRC, VPNPD-91-112, "Status Update Electrical Distribution System Functional Inspection Point Beach Nuclear Plant Units 1 and 2," dated March 28, 1991.

3. NRC Safety Evaluation Dated September 17, 1986, "Safety Evaluation of Topical Report (WCAP-10858)," "AMSAC Generic Design Package."

4. WE Letter to NRC, "Additional Response To NRC Generic Letter 81-14," Point Beach Nuclear Plant, Units 1 and 2, dated May 4, 1982.

5. NRC Letter, Status Report and Technical Evaluation Report, "Seismic Qualification Of The Auxiliary Feedwater System," Point Beach Nuclear Plant Units 1 and 2, dated January 16, 1985.

6. NRC Safety Evaluation, Amendment Nos. 45/50 to Facility Operating License Nos. DPR-24 and DPR-27 for the Point Beach Nuclear Plant, Units 1 and 2, "Low Temperature Overpressure Mitigating Systems," dated May 20, 1980.

7. NRC Letter, "NUREG-0737 Item II.B.1, Reactor Coolant System Vents - Point Beach Nuclear Plant Units 1 And 2," dated September 22, 1983.

8. WE Letter to NRC, "Reactor Coolant System Gas Vent System Point Beach Nuclear Plant, Units 1 and 2," dated June 18, 1982.

9. NRC Safety Evaluation, Addendum No. 5 to the Safety Evaluation in the Matter of Point Beach Nuclear Plant Units 1 and 2, dated November 2, 1971.

10. NRC Safety Evaluation, Amendment Nos. 35/41 to Facility Operating License Nos. DPR-24 and DPR-27 for the Point Beach Nuclear Plant, Units 1 and 2, "Modification of The Spent Fuel Storage Pool," dated April 4, 1979.

11. WE Letter to NRC, "Reactor Vessel Overpressurization," Point Beach Nuclear Plant, Units 1 and 2, dated December 20, 1976.

12. NRC Safety Evaluation, "Main Steam Line Break with Continued Feedwater Addition," Point Beach Nuclear Plant, Units 1 and 2, dated October 8, 1982.

13. WE Letter to NRC, "Final Resolution of Generic Letter 81-14 Seismic Qualification of Auxiliary Feedwater System," Point Beach Nuclear Plant, Units 1 And 2, dated April 26, 1985.

14. NRC Safety Evaluation, "Seismic Qualification of the Auxiliary Feedwater System," Point Beach Nuclear Plant Units 1 And 2, dated September 16, 1986.

15. WE Letter to NRC, "Seismic Qualification of the Auxiliary Feedwater System," Point Beach Nuclear Plant Units 1 and 2, dated December 15, 1982.

16. VPNPD-93-171, "Design Summary for the Installation of Two additional Emergency Diesel Generators - Point Beach Nuclear Plants, Unit 1 and 2," dated September 24, 1993 and attached Report REP-0026, "PBNP Diesel Project Design Submittal," Revision 0, dated September 21, 1993.

17. NRC Safety Evaluation 94-003, "Emergency Diesel Generator Addition Project, Point Beach Nuclear Plant," October 24, 1994.

18. US NRC Generic Letter 87-02, USI A-46 Resolution, Seismic Evaluation Report, Revision 1, dated January 1996.

19. NRC SE, "Response to Supplement No. 1 to Generic Letter 87-02 for the Point Beach Nuclear Plant, Units 1 and 2," dated July 7, 1998.

20. US NRC SE, "Amendment No. 240 to Renewed Facility Operating License No. DPR-24 and Amendment No. 244 to Renewed Facility Operating License No. DPR-27, NextEra Energy Point Beach, LLC, Point Beach Nuclear Plant, Units 1 and 2, Docket Nos. 50-266 and 50-301," dated April 14, 2011.

Insert new references 21 - 27

INSERT

A.5.10 CONSTRUCTION TRUSS DESIGN CODE NONCONFORMANCE RESOLUTION

The trusses used to form the containment concrete domes are described in Section 5, Containment System Structure. The truss structures provided support for the containment dome liner plates and initial concrete pour during station construction. After the initial concrete pour cured, the structures were lowered approximately three inches, no longer providing structural support to the dome. The trusses were then used as an attachment point for containment spray piping, containment air recirculation cooling system (VNCC) ductwork, post-accident containment ventilation (PACV) piping, and miscellaneous lights and associated conduits. An initial analysis of seismic adequacy was performed by the construction vendor. Subsequent analyses and design drawing verification activities identified legacy code nonconformances associated with the seismic response of the trusses and the containment spray piping and pipe supports. In addition, clearance between the trusses and the containment liner at certain locations around the containment circumference were found to be insufficient such that a design basis thermal event or seismic event could result in contact between the trusses and the containment liner due to thermal expansion or truss deflection from applied seismic loads, resulting in code nonconformances for the containment liner/structure. The legacy nonconformances were identified in both Units 1 and 2.

A risk-informed analysis was performed to determine the risk associated with acceptance of the trusses in the as-built configuration considering the occurrence of a seismic or thermal event (Reference 21). A series of engineering calculations were performed to support the risk-informed License Amendment Request (Reference 21). The engineering calculations used alternate evaluation methods and acceptance criteria, as the evaluated structures/components did not meet the original design criteria. The calculations determined the seismic and thermal fragility in support of the risk assessment. The seismic analysis determined one containment spray pipe support in Unit 1, SI-301R-1-H202, required modification. A modification was proposed for Unit 1 to improve clearances between the construction trusses and the containment liner. The clearance modification would result in stress reduction and a configuration bounded by the Unit 2 thermal fragility analysis. The supporting calculations demonstrated that following completion of the Unit 1 truss and containment spray pipe support modifications, structural integrity, i.e., the ability to support carried loads and not interfere with supported equipment functions, was maintained in both Units with adequate margin.

A License Amendment Request (Reference 21) was submitted to accept the low risk code nonconformances following completion of the Unit 1 modifications (Reference 23). Planned modifications that provide protection to increase availability of the reactor coolant system feed and bleed capability were credited as providing additional safety margin (Reference 21) (Reference 24) (Reference 25).

The risk informed resolution includes implementation of revised thermal and seismic limits to initiate assessment of the construction trusses, equipment supported by the trusses, and the containment/containment liner, as necessary, for any event exceeding the specified limits. Any event reaching or exceeding the specified limit(s) requires Unit shutdown and inspection and/or analysis to ensure the affected structures/components can withstand a subsequent design basis accident without adversely impacting the SSCs' design function(s).

THERMAL LIMIT	VALUE
Unit 1 maximum containment atmospheric temperature	227°F (Reference 26)
Unit 2 maximum containment atmospheric temperature	236°F (Reference 26)

SEISMIC LIMIT	VALUE
Horizontal peak ground acceleration	0.053g (Reference 27)
Vertical peak ground acceleration	0.045g (Reference 27)

The risk-informed resolution of the code nonconformances was approved by License Amendment Nos. ___ and ___ dated _____ (Reference 22). The License Amendment results in acceptance of the modified configuration of the Unit 1 construction truss and associated equipment following completion of a modification to increase clearance between the trusses and the containment liner, and modification to containment spray pipe support SI-301R-1-H202, and accepts the current condition of the Unit 2 construction truss and associated equipment.

New References for Section A.5.11:

21. License Amendment Request 278, Risk-Informed Approach to Resolve Construction Truss Design Code Nonconformances, dated March 31, 2017

22. US NRC Safety Evaluation, "Amendment No. XXX to Renewed Facility Operating License No. DPR-24 and Amendment No. XXX to Renewed Facility Operating License No. DPR-27, NextEra Energy Point Beach, LLC, Point Beach Nuclear Plant, Units 1 and 2, Docket Nos. 50-266 and 50-301," dated [Month] [Day], [Year]

23. EC 282198, Modification to Unit 1 Dome Truss to Increase Available Liner Gap; includes SI-301R-1-H202 modification

24. EC 285145, Unit 1 Backup Pneumatic Supply to the PORVs

25. EC 284214, Unit 2 Backup Pneumatic Supply to the PORVs

26. Calculation 11Q0060-C-036, Thermal Evaluation of Units 1 and 2 Containment Dome Trusses for Lesser Events

27. Calculation 11Q0060-C-037, Seismic Evaluation of Units 1 and 2 Containment Dome Trusses for Lesser Events

ENCLOSURE 4

NextEra Energy Point Beach, LLC

License Amendment Request (LAR) 278

Probabilistic Risk Assessment Evaluation PBN-BFJR-17-019

(54 pages follow)

POINT BEACH UNITS 1 & 2
CONSTRUCTION TRUSS
PRA EVALUATION

PBN-BFJR-17-019

Prepared by:	Ted Kulczycky Ted Kulczycky	PRA	3/30/17
Prepared by:	Ted Kulczycky for Jim Moody Jim Moody	PRA	3/30/17
Reviewed by:	S. E. Guokas Stan Guokas	PRA	3/30/17
Reviewed by:	Richard Turcotte Rich Turcotte	PRA *Independent Reviewer*	3/30/17
Approved by:	Anil K. Julka Anil Julka	PRA	3/30/17

Table of Contents

1.0 INTRODUCTION

This evaluation supports a risk informed license amendment request (LAR) that demonstrates the Point Beach Unit 1 and Unit 2 Construction Truss (CT) configurations meet the requirements of Regulatory Guide (RG) 1.174[1] which allows for plant changes as long as the increase in ΔCDF and ΔLERF is small. This evaluation also provides the information required by RG 1.177[2] for risk-informed LARs and documents conformance to the technical standards outlined in RG 1.200[3].

1.1 BACKGROUND

The current CT configuration does not conform to the design criteria for seismic or thermal events. The CT is a legacy structure originally used to support the containment dome during its construction but is no longer in contact with the containment dome. The CT is used to support the safety related containment spray (CS) ring headers, post-accident containment ventilation (PVAC) system, a portion of the containment ventilation ductwork (VNCC) and other, non-safety related, equipment such as lighting and miscellaneous conduit.

Key CT design and configuration issues:

- The CT and the CS piping and supports do not meet original design code criteria for design basis earthquake and thermal[4] transients.
- Some clearances between the CT and containment liner are smaller than expected and may not prevent overstress due to contact with the liner resulting from thermal expansion under Design Basis Accident (DBA) temperatures.

Reference 15 describes in detail the technical issues of the structural non-conformances relating to the CT and attached components.

BASIS FOR OPERABILITY:

SEISMIC

Prompt operability determination (POD) 02131629-02 concluded that the Unit 1 and Unit 2 CT will remain stable and will not experience a catastrophic failure from a ground motion response spectra (Housner) based seismic event.

THERMAL

Unit 1: POD 01962836-01 concluded that the Unit 1 CT, while non-conforming to the design code of record, would maintain integrity and supported components remained acceptable for the design basis thermal loading. Applied loads resulting in plastic strain develop, which is a challenge, but overall integrity is maintained/liner is not breached. Containment integrity is maintained, and the containment spray piping remains acceptable.

---

[1] *REGULATORY GUIDE 1.174, Revision 2, An Approach For Using Probabilistic Risk Assessment In Risk-Informed Decisions On Plant Specific Changes to the Licensing Basis.*
[2] *RG 1.177, "An Approach for Plant-Specific, Risk-Informed Decisionmaking: Technical Specifications."*
[3] *RG 1.200, "An Approach for Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities."*
[4] *CT does not meet original design code criteria for thermal, however CS piping and supports are acceptable for thermal load.*

Unit 2: POD 01986553-01 concluded that the configuration for Unit 2 CT was bounded by the configuration of the U1 CT, and the CT, while non-conforming to the design code of record, would maintain integrity and supported components remained acceptable for the design basis thermal loading. Containment integrity is maintained, and the containment spray piping remains acceptable.

1.2 CT MODIFICATION

Trimming is needed at several Unit 1 CT top chord first panel point locations to reduce the contact load between the CT and containment liner during a DBA thermal condition and to reduce stresses in truss components. The Unit 2 CT will not be modified. Reference 15 provides detailed information regarding this modification.

1.3 SUMMARY OF ISSUES EVALUATED

This evaluation provides quantitative and qualitative risk insights for the following issues:

- **SEISMIC.** The CT is assumed to be within normal operating temperature (below 120 F) prior to the seismic event. CT failure is assumed to occur at the initiation of the seismic event, at T=0.
- **THERMAL.** The CT failure due to thermal growth occurs sometime after the thermal transient initiating event, at T > 0. Thermal initiating events that can initiate this challenge include steam line break, large LOCA, medium LOCA, small LOCA, and feedwater line break. Steam line break governs the containment design temperature of 286°F [Ref 1].

1.4 RISK CRITERIA

CT risk has been evaluated against criteria from RG 1.174[5] which allows licensing basis plant changes as long as the increase in risk (△CDF and △LERF) is small. Figures 4 and 5 (reproduced in Figures 1 and 2 below) in the RG illustrate the acceptance guidelines and show that a change in CDF between 1E-6 and 1E-5 per year is acceptable as long as the total plant CDF for all hazards does not equal or exceed 1E-4. LERF criterion is similar except an order of magnitude less.

FIGURE 1: NRC RG 1.174 FIGURE 4

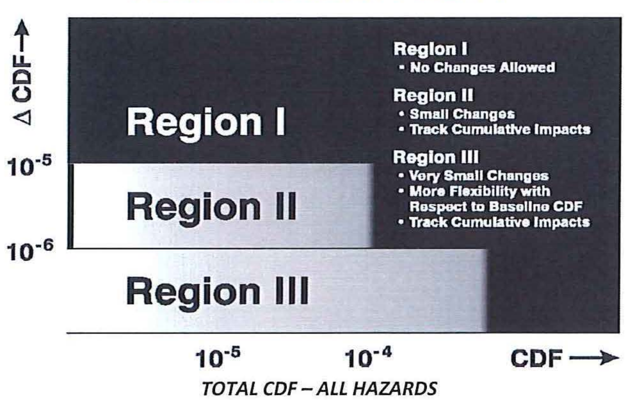


FIGURE 2: NRC RG 1.174 FIGURE 5

[5] *RG 1.174, An Approach For Using Probabilistic Risk Assessment In Risk-Informed Decisions On Plant Specific Changes To The Licensing Basis.*

1.5 TOTAL PLANT CDF

The plants "ALL-HAZARDS" risk establishes the applicable RG 1.174 region and thereby the risk criteria applied in this evaluation. The following table summarizes the "ALL-HAZARDS" risk results for Point Beach [Ref 19].

Table 1: Point Beach ALL HAZARDS PRA Results				
PRA Scope	**CDF** *(1/Rx Yr)*		**LERF** *(1/Rx Yr)*	
	Unit 1	**Unit 2**	**Unit 1**	**Unit 2**
Internal Events at Power	5.1E-06	5.1E-06	3.7E-08	3.6E-08
Internal Floods at Power	3E-07	3E-07	2E-08	2E-08
Internal Fire at Power	5.9E-05	6.9E-05	9.0E-07	1.1E-06
High Winds at Power	1.74E-06	1.16E-06	5.73E-08	5.25E-08
Seismic at Power	6.24E-06	6.24E-06	1.21E-06	1.21E-06
Other Hazards	<1E-06	<1E-06	<1E-07	<1E-07
Total	**7.3E-05**	**8.3E-05**	**2.3E-06**	**2.5E-06**

The "ALL HAZARD" CDF totals for both units are below 1E-4 and thereby establish △CDF <1E-5 as the acceptable criterion for this application. The LERF totals are less than 1E-5 and thereby establish <1E-6 as the acceptable △LERF criterion.

1.6 PRA MODEL SCOPE

The ASME Standard [Ref 10], Section 6-2.3[6] has guidance on screening hazards without a full-scope PRA. Options include:

- Demonstrably conservative analysis
- Bounding PRA
- Limited PRA
- Intermediate Approach

The two options used in this evaluation are:

- Bounding PRA
- Demonstrably conservative analysis

The bounding and demonstrably conservative analyses show that △CDF and △LERF are acceptably low for hazards challenging the CT design. The inputs used in these analyses include hazard analysis, fragility analysis, and systems analysis, human-reliability analysis, and accident-sequence analysis.

[6] *Reference 10, **6-2.3 SCREENING CRITERIA,** There are three fundamental screening criteria embedded in the requirements here, as follows. An event can be screened out either*

(a) if it meets the criteria in the NRC's 1975 Standard Review Plan (SRP) [6-2] or a later revision; or

(b) if it can be shown using a demonstrably conservative analysis that the mean value of the frequency of the design-basis hazard used in the plant design is less than ~10^{-5}/yr and that the conditional core damage probability is <10^{-1}, given the occurrence of the design-basis hazard event; or

(c) if it can be shown using a demonstrably conservative analysis that the CDF is 10^{-6}/yr.

1.6.1 BOUNDING ANALYSIS

The bounding analysis assumes CT overstress always leads to core damage. No credit is provided for mitigating systems and operator actions. As such neither a full-scope nor partial PRA model is needed to perform this assessment. Only hazard and fragility data are used in this analysis.

The assumption that a CT collapse will always occur when the structure is overstressed and that this will always lead to core damage is represented by setting the conditional core damage probability (CCDP) to 1.0. This provides an upper limit of the risk metrics that inherently include the worst credible outcome of all known possible outcomes of a CT overstress and as such this assumption is bounding in terms of the potential outcome and the likelihood of outcome. This approach is consistent with RG 1.174 which states that a PRA should include a full understanding of the impacts of the uncertainties through either a formal quantitative analysis or a simple bounding or sensitivity analyses. The simple bounding case and sensitivity analyses applied in this evaluation address all known uncertainties.

A variation of this case was quantified by convolving the IPEEE seismic PRA with the seismic bounding results. Section 2.1.4 describes the method used and results.

1.6.2 DEMONSTRABLY CONSERVATIVE ANALYSIS

To assess the level of conservatism of the bounding analysis, a "demonstrably conservative analysis" was developed using a limited scope PRA. Demonstrably conservative analysis uses assumptions such that the assessed outcome will be conservative relative to the expected outcome [Ref 10].

This analysis applied more realistic assumptions, assuming it is unlikely that a CT collapse will always lead to core damage; applying a more realistic assumption that CCDP is less than 1.0. However, the conservative assumption that the CT will collapse if overstressed was retained. Seismic and thermal events are evaluated separately.

For this assessment the following aspects of a CT failure and its consequences were studied:

- How the CT fails when overstressed
- Trajectory of components from the failed CT
- Vulnerability of risk significant components to falling CT debris
- Location and robustness of the barriers that would protect critical components from the falling CT debris.

The results of this assessment [Ref 3] identified systems, structures, and components (SSCs) that are likely to survive a CT failure and operator actions that remain viable under seismic or thermal (e.g. LOCA) transients. Although this analysis is more realistic than the bounding analysis, conservative assumptions were applied to assure key uncertainties are addressed or bounded, for example:

- The CT structure is always assumed to immediately generate falling debris when overstressed.
- CT debris targeting critical SSCs are assumed to be oriented in a way that maximizes damage to targeted SSCs.
- Performance shaping factors, factors affecting operator actions, were increased to address additional stress and concurrent critical actions that would reduce the reliability of operator actions.

1.7 RESULTS SUMMARY

The results of these analyses show the modified Unit 1 CT and current Unit 2 CT △CDF and △LERF are within the RG 1.174 criteria. △LERF is based on an assumed LERF/CD ratio of 0.2. This is the conditional probability that a large early release will occur given that core damage has occurred, also referred to as conditional large early release probability (CLERP). Section 6, LERF, provides the basis for assuming a CLERP of 0.2.

1.7.1 SEISMIC RISK

The seismic risk is acceptable based on Region II criteria for both units for Case 1a. Case 1b meets the more restrictive RG 1.174 Region III criteria, △CDF <1E-6 and △LERF <1E-7 versus Region II criteria of △CDF <1E-5 and △LERF <1E-6.

Case		△CDF	△LERF
1a	**Bounding CT convolved with Seismic IPEEE** *[section 2.1.4]*	1.88E-06	3.74E-07
1b	**Demonstrably Conservative convolved with Seismic IPEEE** *[section 2.1.4]*	9.38E-08	1.88E-08

1.7.2 THERMAL RISK

The thermal risk is acceptable based on Region II criteria for both units for the bounding case. Both the bounding and demonstrably conservative cases meet the more restrictive RG 1.174 Region III criteria.

Case		△CDF	△LERF
2	**Bounding** *[section 2.2]*	7.30E-07	1.46E-07
3	**Demonstrably Conservative** *[section 5]*	4.02E-08	8.04E-09

1.7.3 TOTAL RISK

Total risk is acceptable based on Region II criteria for both units for the bounding case. The demonstrably conservative case meets the more restrictive RG 1.174 Region III criteria. A CLERP of 1.0 for the demonstrably conservative case meets Region II LERF criterion. That is, if a large early release always occurs when there is core damage, Region II criterion of <1E-06 for △LERF is still met and as such bounds all uncertainties related to potential releases or likelihood of release.

Case	△CDF	△LERF	Maximum CLERP Region II =<1E-06	Maximum CLERP Region III =<1E-07
Bounding with convolved IPEEE, *Cases 1a + 2*	2.61E-06	5.22E-07	0.38	0.04
Demonstrably Conservative convolved with IPEEE, *Cases 1b + 3*	1.34E-07	2.68E-08	1.00	0.72

Adding the △CDF and △LERF values in the table above to the "ALL HAZARDS" total (table 1) will not change the RG 1.174 criteria applicable to this evaluation.

1.7.4 QUANTIFICATION SUMMARY

The table below summarizes all the cases quantified in this evaluation and each case is referenced to its respective section in this evaluation. The results are coded with the RG 1.174 region that the resultant value complies with.

RG 1.174	ΔCDF	ΔLERF
REGION I	>1.0E-05	>1.0E-06
REGION II	>1.0E-06 ⇨ 1.0E-05	>1.0E-07 ⇨ 1.0E-06
REGION III	⇨1.0E-06	⇨ 1.0E-07

	CASE		ΔCDF	ΔLERF *CLERP=0.2*
SEISMIC	1	**Bounding** *[section 2.1]*	**2.69E-06**	**5.38E-07**
SEISMIC	1a	**Bounding CT convolved with Seismic IPEEE** *[section 2.1.4]*	**1.88E-06**	**3.74E-07**
SEISMIC	2	**Demonstrably Conservative** *[section 5]*	**1.34E-07**	**2.68E-08**
SEISMIC	2a	**Demonstrably Conservative, convolved with Seismic IPEEE** *[section 5.2.1]*	**9.38E-08**	**1.88E-08**
THERMAL	3a	**Bounding** *[section 2.2]*	**7.30E-07**	**1.46E-07**
THERMAL	3b	**Bounding Sensitivity (all SLBs)** *[Section 2.2.6]*	**1.59E-06**	**3.18E-07**
THERMAL	4	**Demonstrably Conservative** *[section 5]*	**4.02E-08**	**8.04E-09**
TOTAL	**Bounding,** Cases 1 + 3a		**3.42E-06**	**6.84E-07**
TOTAL	**Bounding with convolved IPEEE**, Cases 1a + 3a		**2.61E-06**	**5.22E-07**
TOTAL	**Demonstrably Conservative,** Cases 2+ 4		**1.74E-07**	**3.48E-08**
TOTAL	**Demonstrably Conservative with convolved IPEEE,** Cases 2a + 4		**1.34E-07**	**2.68E-08**

2.0 BOUNDING ANALYSIS

The bounding analysis assumes CT overstress always leads to core damage. The assumption that a CT collapse will always occur when the structure is overstressed and that this will always lead to core damage is represented by setting the conditional core damage probability (CCDP) to 1.0. This provides an upper limit of the risk metrics that inherently include the worst credible outcome of all known possible outcomes of a CT overstress and as such this assumption is bounding in terms of the potential outcome and the likelihood of outcome. This approach is consistent with RG 1.174 which states that a PRA should include a full understanding of the impacts of the uncertainties through either a formal quantitative analysis or a simple bounding or sensitivity analyses. The simple bounding case and sensitivity analyses applied in this evaluation address all known uncertainties.

No credit is provided for mitigating systems and operator actions. As such neither a full-scope nor partial PRA model is needed to perform this assessment. Only hazard and fragility data are applied. Other concurrent failures that would independently lead to core damage and reduce the core damage contribution of a CT failure were not included. During a seismic event many concurrent but independent failures are likely, but much less so during a thermal transient; omitting these elements is conservative.

A variation of this case was quantified by convolving the IPEEE seismic PRA with the seismic bounding results. Section 2.1.4 describes the method used and results.

Seismic and thermal events are assumed not to occur concurrently; they are independent initiating events.

The following sections evaluate the seismic and thermal transient impact on the CT. Seismic and thermal events are evaluated separately.

2.1 SEISMIC ANALYSIS

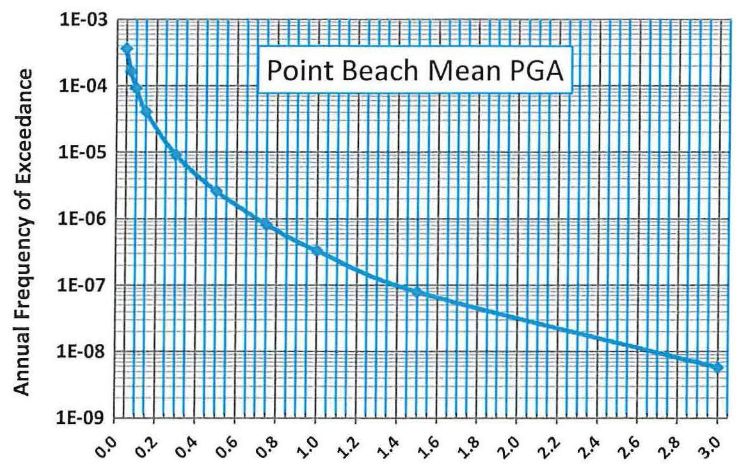
This section provides the seismic analysis results along with inputs and methods used to perform the analysis.

2.1.1 SEISMIC HAZARD

The seismic evaluation uses site specific seismic hazard data to develop discrete seismic initiating events that are convolved with the fragility data to calculate ΔCDF and ΔLERF.

The following table provides the Point Beach seismic hazard data from Point Beach Seismic Hazard and Screening Report (Ref 4). The mean values from this table are used to calculate the frequencies for the seismic initiating events used in this evaluation.

Table 2: POINT BEACH PEAK GROUND ACCELERATION (PGA)			
PGA	5%	Mean	95%
0.050	*5.35E-05*	**3.69E-04**	*1.20E-03*
0.075	*1.84E-05*	**1.67E-04**	*5.50E-04*
0.100	*8.23E-06*	**9.41E-05**	*3.14E-04*
0.150	*2.60E-06*	**4.14E-05**	*1.42E-04*
0.300	*3.05E-07*	**9.28E-06**	*3.28E-05*
0.500	*5.27E-08*	**2.66E-06**	*1.01E-05*
0.750	*1.11E-08*	**8.43E-07**	*3.37E-06*
1.000	*3.42E-09*	**3.36E-07**	*1.40E-06*
1.500	*5.12E-10*	**8.07E-08**	*3.52E-07*
3.000	*9.11E-11*	**5.89E-09**	*2.68E-08*



Source: NRC 2014-0024, Appendix A, Table A1-a. Mean and Fractile Seismic Hazard Curves for PGA at Point Beach

SEISMIC INITIATING EVENT frequencies were developed using EPRI FRANX software [Ref 2] based on the mean hazard data from Table 2. Attachment D provides the FRANX Hazard Editor output. Ten bins of seismic initiating events were developed. Ten bins provided the same results as 20 bins; the results were no longer stable when 8 or fewer bins were used. The resulting initiating events bins are listed in Table 3A. These events are convolved with their respective seismic conditional failure probabilities to estimate the mean annual frequency of occurrence as described in the next section.

Table 3A: SEISMIC HAZARD		
Seismic Initiating Events		Hazard Frequency
ID	PGA Range	
%G01	0.05g to <0.12g	3.04E-04
%G02	0.12g to <0.23g	4.82E-05
%G03	0.23g to <0.34g	1.01E-05
%G04	0.34g to <0.45g	3.36E-06
%G05	0.45g to <0.56g	1.54E-06
%G06	0.56g to <0.67g	7.72E-07
%G07	0.67g to <0.78g	4.34E-07
%G08	0.78g to <0.89g	2.56E-07
%G09	0.89g to <1g	1.54E-07
%G10	>1g	3.36E-07

2.1.2 SEISMIC FRAGILITY ANALYSIS

The conditional failure probability used to calculate the annual frequency of occurrence is a function of fragility. The annual frequency of occurrence is equated to CDF for the bounding analysis as follows:

CDF = Annual Frequency of Occurrence = Hazard Frequency * Conditional Failure Probability [fragility]

The fragility calculation is based on guidance from Reference 9, section *4.1.1 Fragility Model.* The following excerpts from that document provide background for derivation, calculation and application of fragility:

> *With perfect knowledge of the failure mode and parameters describing the ground acceleration capacity (i.e., only accounting for the random variability, β_R), the*

conditional probability of failure, f_O, for a given peak ground acceleration level, a, is given by Equation 2-2:

$$\textbf{Conditional Probability of Failure} = f_o = \phi\left[\frac{ln\left(\frac{a}{A_m}\right)}{\beta_R}\right]$$

*Where **Φ [ ]** is the standard Gaussian (normal) cumulative distribution of the term in brackets.*

*The relationship between f_o and **a** is the median fragility curve for a component with a median ground acceleration capacity A_m.*

The mean fragility curve is obtained using Eq. 2-2 but replacing β_R with the composite variability:

$$\beta_C = (\beta_R^{\ 2} + \beta_U^{\ 2})^{1/2}$$

Where β_U is the modeling uncertainty.

In the IPEEE program, only a point estimate (mean value) of CDF was required, thus single mean fragility curves and the mean seismic hazard curve were convolved to calculate the unconditional probability of failure of SSCs.

Based on this guidance the following equation was used to calculate the mean fragilities for each of the discrete initiating events applied to this analysis:

$$\textbf{Conditional Failure Probability} = \phi\left[\frac{ln\left(\frac{a}{A_m}\right)}{\beta_C}\right] \qquad \textbf{\textit{Equation 1}}$$

Where:

Φ is the standard Gaussian (normal) cumulative distribution
***a** = PGA level (based on the geometric mean of the upper and lower range of the hazard, the square root of their product, was calculated for each initiating event.)*
A_m = CT median acceleration capacity
$\beta_c = (\beta_r^{\ 2} + \beta_u^{\ 2})^{1/2}$ = composite or mean standard deviation
β_r = logarithmic standard deviation of the capacity and represents variability due to randomness in earthquake and structural characteristic
β_u = logarithmic standard deviation of median capacity which represents uncertainty in models.

Equation 1 was used to calculate the Conditional Failure Probability, fragility, related to a catastrophic CT overstress condition that directly leads to core damage. Failure is defined as an overstress condition only. The structural calculations did not include an assessment of the consequences of overstress, i.e. the extent to which overstress would affect the stability of the CT or its components, or the likelihood of deconstruction, collapse, etc.

A_m, β_r, β_u, and β_c values were obtained from structural calculation 11Q0060-C-028 [Ref 6]:

TABLE 3B	A_m	β_u	β_r	β_c
Unit 1Thermal Mod, Unit 2 Unmodified	**0.42**	**0.32**	**0.24**	**0.40**
All Design Basis Mods	**0.53**	**0.32**	**0.24**	**0.40**

2.1.3 SEISMIC BOUNDING ANALYSIS

The seismic hazard and fragility data from the previous section are applied to the bounding case. The conditional failure probability is calculated using Equation 1 and data from Table 3A. Since the bounding case assumes CCDP=1.0, CDF is set equal to Annual Frequency of Occurrence which is calculated as follows:

CDF = Annual Frequency of Occurrence = Hazard Frequency * Conditional Failure Probability

The following tables provide results for the post LAR as built CT and the fully design compliant CT:

TABLE 4: SEISMIC HAZARD ANNUAL FREQUENCY OF OCCURRENCE

Unit 1 with THERMAL MODIFICATIONS[7], UNMODIFIED Unit 2

Seismic Initiating Events		Hazard Frequency	a	A_m	β_c	Conditional Failure Probability	Annual Frequency of Occurrence
ID	**PGA Range**						
%G01	0.05g to <0.12g	3.04E-04	0.0775g	0.42	0.40	1.20E-05	3.63E-09
%G02	0.12g to <0.23g	4.82E-05	0.1661g	0.42	0.40	1.02E-02	4.91E-07
%G03	0.23g to <0.34g	1.01E-05	0.2796g	0.42	0.40	1.55E-01	1.56E-06
%G04	0.34g to <0.45g	3.36E-06	0.3912g	0.42	0.40	4.30E-01	1.44E-06
%G05	0.45g to <0.56g	1.54E-06	0.5020g	0.42	0.40	6.72E-01	1.04E-06
%G06	0.56g to <0.67g	7.72E-07	0.6125g	0.42	0.40	8.27E-01	6.39E-07
%G07	0.67g to <0.78g	4.34E-07	0.7229g	0.42	0.40	9.13E-01	3.96E-07
%G08	0.78g to <0.89g	2.56E-07	0.8332g	0.42	0.40	9.57E-01	2.45E-07
%G09	0.89g to <1g	1.54E-07	0.9434g	0.42	0.40	9.78E-01	1.51E-07
%G10	>1g	3.36E-07	1.1000g	0.42	0.40	9.92E-01	3.33E-07
						CDF = TOTAL =	**6.30E-06**

TABLE 5: SEISMIC HAZARD ANNUAL FREQUENCY OF OCCURRENCE

All Design Basis Mods

Unit 1 and Unit 2 MODIFIED to FULLY MEET SEISMIC and THERMAL DESIGN REQUIREMENTS

Seismic Initiating Events		Hazard Frequency	a	A_m	β_c	Conditional Failure Probability	Annual Frequency of Occurrence
ID	**PGA Range**						
%G01	0.05g to <0.12g	3.04E-04	0.0775	0.53	0.40	7.68E-07	2.33E-10
%G02	0.12g to <0.23g	4.82E-05	0.1661	0.53	0.40	1.86E-03	8.97E-08
%G03	0.23g to <0.34g	1.01E-05	0.2796	0.53	0.40	5.49E-02	5.55E-07
%G04	0.34g to <0.45g	3.36E-06	0.3912	0.53	0.40	2.24E-01	7.52E-07
%G05	0.45g to <0.56g	1.54E-06	0.502	0.53	0.40	4.46E-01	6.87E-07
%G06	0.56g to <0.67g	7.72E-07	0.6125	0.53	0.40	6.41E-01	4.95E-07
%G07	0.67g to <0.78g	4.34E-07	0.7229	0.53	0.40	7.81E-01	3.39E-07
%G08	0.78g to <0.89g	2.56E-07	0.8332	0.53	0.40	8.71E-01	2.23E-07
%G09	0.89g to <1g	1.54E-07	0.9434	0.53	0.40	9.25E-01	1.42E-07
%G10	>1g	3.36E-07	1.1	0.53	0.40	9.66E-01	3.25E-07
						CDF = TOTAL =	**3.61E-06**

Assuming a CCDP of 1.0, the △CDF is 2.69E-06 when compared to a CT that meets GMRS-based seismic event requirements:

	MODIFICATIONS	CDF	*A – B =* △**CDF** *[CCDP = 1.0]*
A	Thermal Mod Unit 1, Unit 2 Unmodified	**6.30E-06**	**2.69E-06**
B	All Design Basis Mods (Base CDF)	**3.61E-06**	

This value meets the Region II criterion of <1E-5 △CDF.

[7] *Unit 1 thermal modifications result in a configuration that is bounded by to Unit 2 for applied thermal loads.*

2.1.4 IPEEE and BOUNDING CT CONVOLUTION

The bounding assessment in section 2.1.3 omits seismic risk associated with SSC contributors independent of the CT. Omitting this consideration overestimates CT risk. In some seismic sequences core damage would occur irrespective of CT failure.

Earthquakes simultaneously affect multiple redundant components, leading to many different concurrent failures and accident sequences, many of which would not involve the CT. Some of the accident sequences not involving the CT may dwarf or subsume the risk of components damaged by a CT failure, i.e. they may have a much higher CDF for a given seismic hazard. A more accurate characterization of the CT risk can be obtained by combining the IPEEE seismic and the CT bounding results.

The convolution[8] applied the updated Point Beach GMRS as documented in ML14090A275 and accepted by the NRC in ML15211A593. This is the same data used to develop the CT fragility curve and as such allows convolving the CT CDF and IPEEE CDF values; thereby integrating the CT with the IPEEE.

The following information summarizes the data used to convolve the CT with the IPEEE.

Case		**Am**	**Bu**	**Br**	**Bc**	**HCLPF**
1	*CT U1 Mod, U2 As is*	0.42	0.32	0.24	0.40	0.17
2	*CT All Design Basis Mods*	0.53	0.32	0.24	0.40	0.21
3	**IPEEE**	0.45			0.45	0.16

Fragilities and CDFs were developed for each of these three cases, and then the IPEEE data was convolved and logically "ORed" (*) with each of the two CT cases, cases 1 and 2. The results are provided in the tables below.

Case		**CDF**	**CT * IPEEE**	**△CDF**
1	*CT U1 Mod, U2 As is*	**6.30E-06**	Case 1 * Case 3 = **9.36E-06**	**1.88E-06**
2	*CT All Design Basis Mods*	**3.61E-06**	Case 2 * Case 3 = **7.49E-06**	
3	**IPEEE**	**5.97E-06**		

Where: ***[Case 1 * Case 3] – [Case 2 * Case 3] = △CDF***

9.36E-06 - 7.49E-06 = ***1.88E-06***

A △CDF of 1.88E-06 meets the Region II criterion of <1E-05.

The following table provides details of the calculation of fragilities, probabilities, and CDF for each case.

---

[8] *For any given acceleration core damage can occur as a result of CT overstress or a result of a failure not related to a CT failure. There is also the probability that both could concurrently lead to core damage. The logical OR function deletes this overlapping probability, i.e. it avoids double counting core damage probability. The OR function is represented by the following equation: P(A,B) = P(A) + P(B) – P(A)*P(B). In this application the two functions, CT probabilities as a function of acceleration and IPEEE probabilities as a function of acceleration, are convolved, or integrated, using this logical OR function.*

"CT U1 Mod, U2 As-Is" convoluted with IPEEE

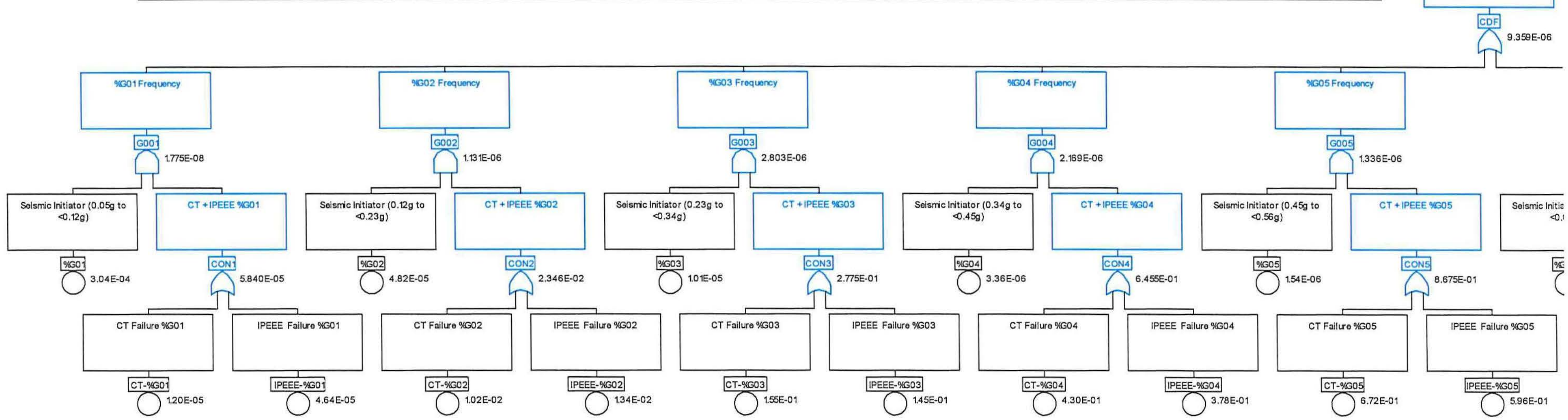
CT U1 Mod, U2 As Is [CASE 1]					IPEEE [CASE 3]		CASE1 + CASE3 - (CASE1*CASE3)	
Hazard		Hazard Frequency	Probability of Failure	Frequency	Probability of Failure	Frequency	Probability of Failure	Frequency
ID	Description							
%G01	Seismic Initiator (0.05g to <0.12g)	3.04E-04	1.20E-05	3.63E-09	4.64E-05	1.41E-08	5.83E-05	1.77E-08
%G02	Seismic Initiator (0.12g to <0.23g)	4.82E-05	1.02E-02	4.91E-07	1.34E-02	6.45E-07	2.34E-02	1.13E-06
%G03	Seismic Initiator (0.23g to <0.34g)	1.01E-05	1.55E-01	1.56E-06	1.45E-01	1.47E-06	2.77E-01	2.80E-06
%G04	Seismic Initiator (0.34g to <0.45g)	3.36E-06	4.30E-01	1.44E-06	3.78E-01	1.27E-06	6.45E-01	2.17E-06
%G05	Seismic Initiator (0.45g to <0.56g)	1.54E-06	6.72E-01	1.04E-06	5.96E-01	9.18E-07	8.68E-01	1.34E-06
%G06	Seismic Initiator (0.56g to <0.67g)	7.72E-07	8.27E-01	6.39E-07	7.53E-01	5.82E-07	9.57E-01	7.39E-07
%G07	Seismic Initiator (0.67g to <0.78g)	4.34E-07	9.13E-01	3.96E-07	8.54E-01	3.71E-07	9.87E-01	4.28E-07
%G08	Seismic Initiator (0.78g to <0.89g)	2.56E-07	9.57E-01	2.45E-07	9.14E-01	2.34E-07	9.96E-01	2.55E-07
%G09	Seismic Initiator (0.89g to <1g)	1.54E-07	9.78E-01	1.51E-07	9.50E-01	1.46E-07	9.99E-01	1.54E-07
%G10	Seismic Initiator (>1g)	3.36E-07	9.92E-01	3.33E-07	9.76E-01	3.28E-07	1.00E+00	3.36E-07
			Total CDF	**6.30E-06**		**5.97E-06**		**9.36E-06**

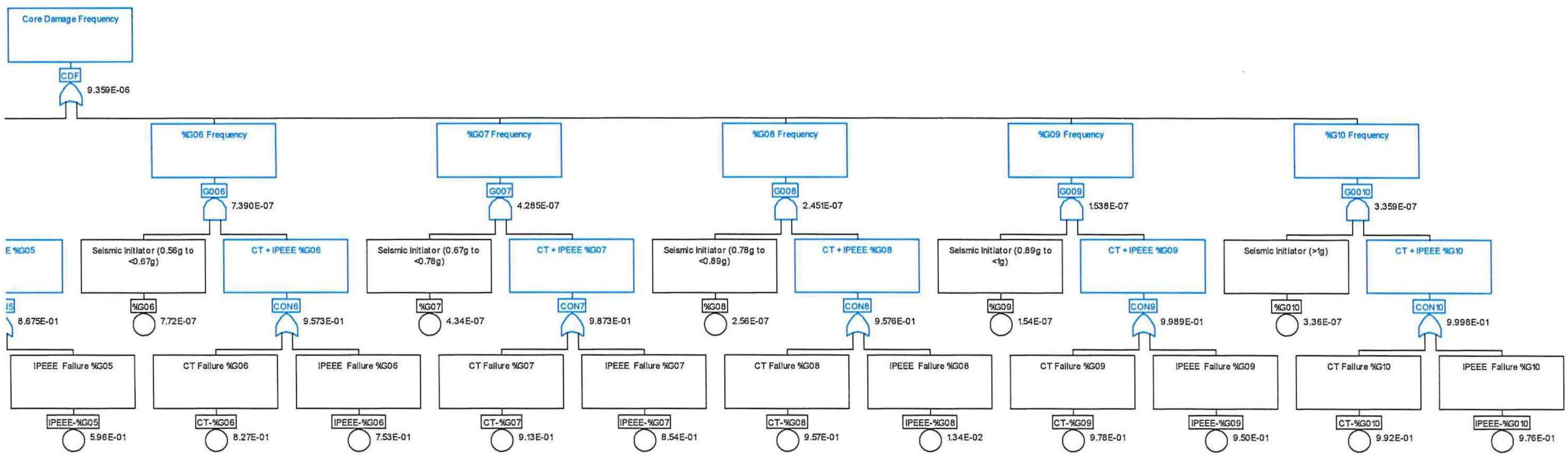
"CT All Design Basis Mods" convoluted with IPEEE

CT All Design Basis Mods [CASE 2]							CASE2 + CASE3 - (CASE2*CASE3)	
%G01	Seismic Initiator (0.05g to <0.12g)	3.04E-04	7.68E-07	2.33E-10	4.64E-05	1.41E-08	4.71E-05	1.43E-08
%G02	Seismic Initiator (0.12g to <0.23g)	4.82E-05	1.86E-03	8.97E-08	1.34E-02	6.45E-07	1.52E-02	7.34E-07
%G03	Seismic Initiator (0.23g to <0.34g)	1.01E-05	5.49E-02	5.55E-07	1.45E-01	1.47E-06	1.92E-01	1.94E-06
%G04	Seismic Initiator (0.34g to <0.45g)	3.36E-06	2.24E-01	7.52E-07	3.78E-01	1.27E-06	5.17E-01	1.74E-06
%G05	Seismic Initiator (0.45g to <0.56g)	1.54E-06	4.46E-01	6.87E-07	5.96E-01	9.18E-07	7.76E-01	1.20E-06
%G06	Seismic Initiator (0.56g to <0.67g)	7.72E-07	6.41E-01	4.95E-07	7.53E-01	5.82E-07	9.12E-01	7.04E-07
%G07	Seismic Initiator (0.67g to <0.78g)	4.34E-07	7.81E-01	3.39E-07	8.54E-01	3.71E-07	9.68E-01	4.20E-07
%G08	Seismic Initiator (0.78g to <0.89g)	2.56E-07	8.71E-01	2.23E-07	9.14E-01	2.34E-07	9.89E-01	2.53E-07
%G09	Seismic Initiator (0.89g to <1g)	1.54E-07	9.25E-01	1.42E-07	9.50E-01	1.46E-07	9.96E-01	1.53E-07
%G10	Seismic Initiator (>1g)	3.36E-07	9.66E-01	3.25E-07	9.76E-01	3.28E-07	9.99E-01	3.36E-07
			Total CDF	**3.61E-06**		**5.97E-06**		**7.49E-06**

△CDF	**1.88E-06**

CAFTA Fault Tree Illustrating "CT U1 Mod, U2 As-Is" convoluted with IPEEE





2.2 THERMAL ANALYSIS

The yearly frequency of a CT thermal overstress condition is based on convolving the CT thermal fragilities (probability of "failure" as a function of temperature), with the thermal initiating event frequencies. Failure is defined as an overstress condition only. The structural calculations, Ref 12.a and 12.b, did not include an assessment of the consequences of overstress, i.e. the extent to which overstress would affect the stability of the CT or its components, or the likelihood of deconstruction, collapse, etc.

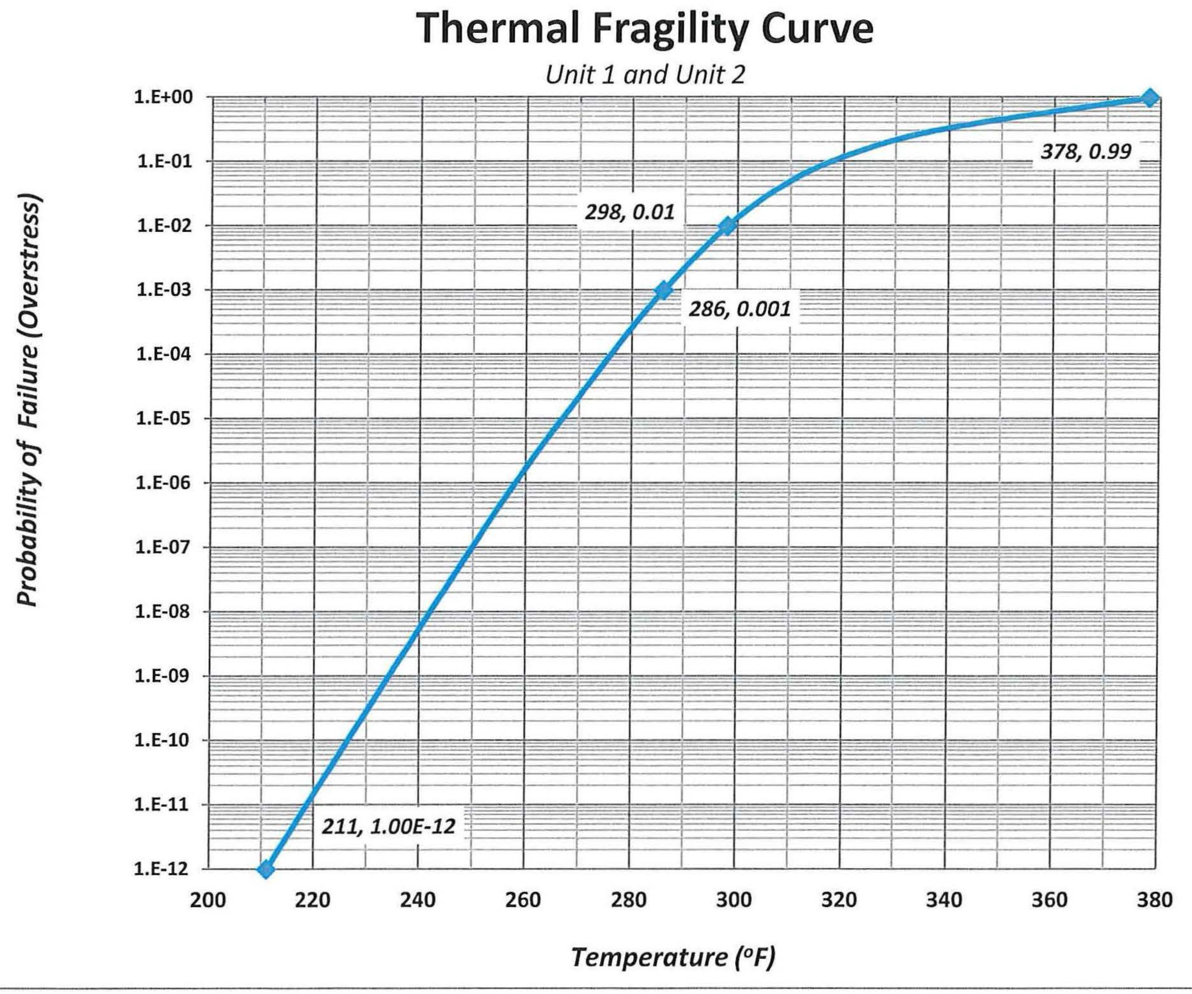
2.2.1 THERMAL FRAGILITY ANALYSIS

The method used to develop the thermal fragility data is described in section 6.4.1.2 of Engineering Evaluation 2017-0008 [Ref 15]. The following table summarizes the resulting fragilities that apply to both units:

Table 6: Thermal Fragility Results

Temperature (°F)	Probability of Failure
211	1E-12
286	0.001
298	0.01
378	0.99

Figure 3



2.2.2 THERMAL INITIATING EVENT FREQUENCY

CT overstress due to thermal growth occurs sometime after the thermal transient initiating event, at T>0. As shown in Figure 3, the probability of CT failure increases with temperature.

This section develops the CT thermal hazard curve based on the total frequency of all applicable thermal initiating events versus the containment air temperatures expected for these events. Total frequency is based on the sum of the frequency of all independent events that result in containment temperatures within the range of the fragility curve (Table 6).

The thermal events considered include:

- Large, Medium and Small LOCAs.
- Steam Line Breaks Inside (SLBI) Containment
- Feed Line Breaks Inside (FLBI) Containment

SLBI is controlling with regard to design basis temperature of 286°F and pressure inside containment [Ref 1]. The design basis SLBI is a 1.4 ft^2 break (limiting size based on the orifice in the line), however a double-ended break is simulated to maximize steam flow until the MSIVs close.

A spectrum of break sizes was considered in developing the set of thermal transients that would result in temperatures within the range of the fragility curve:

- **Design Basis Break (1.4 ft^2)** – this is equivalent to a 16-inch diameter break; much larger than the large LOCA lower bound of 6 inches.
- **Smaller Break (8-inch ~ 0.35 ft^2)** – an 8-inch diameter break size was chosen as another point on the hazard curve for temperature evaluation.
- **3-inch Break (0.05 ft^2)** – This is the lower bound break size that conservatively could challenge the CT.

2.2.2.1 STEAM LINE BREAK INSIDE (SLBI) CONTAINMENT

NUREG/CR-5750 defines a SLBI as a break of one-inch equivalent diameter or more but not greater than 30 inches, maximum pipe diameter in a steam line inside the primary containment.

EPRI 302000079, Rev 3 "Pipe Rupture Frequencies for Internal Flooding Probabilistic Risk Assessments" contains data on main steam line failure rates for different break sizes[9]. Based on this document failure rates were developed for different break sizes and listed in Table 7:

[9] *Table ES-3 "PWR HP Steam" and "PWR FWC CS piping" of the EPRI report*

Table 7: Steam Break Frequencies				
Steam Line Break Inside Containment				
Break size	**U1 (ft)[10]**	**U1 Freq.**	**U2 (ft)[10]**	**U2 Freq.**
>1" to 3"	215	**2.0E-04**	294	**2.8E-04**
>3" to 8"	215	**5.6E-05**	294	**7.7E-05**
>8" to 16"	215	**5.2E-05**	294	**7.1E-05**
>16" to 30"	215	**3.5E-05**	294	**4.3E-05**
Note: these are based on 0.908 pipe wall. Most MS pipe inside containment is 1.125 pipe wall.				

Application of these failure rates to piping inside containment is conservative. For example, the 30-inch main steam pipe outside containment has a pipe wall thickness of 0.908 inches [Ref 8] and most of the 30-inch main steam pipe inside containment has a pipe wall thickness of 1.125 inches, a difference of about 24%. All other factors being equal the failure rate of the thicker pipe is less than the thinner pipe.

2.2.2.2 LOCA

The LOCA initiating event frequencies are obtained from NUREG/CR-6928:

Table 8: LOCA Frequencies			
PRA Designator	**Frequency** *per year*	**PRA Description**	**Data Source**
A	**1.3E-6**	**Large LOCA** – *greater than 6-inch diameter break*	NUREG/CR-6928
S1	**5.1E-4**	**Medium LOCA** - *2 to 6-inch diameter break*	NUREG/CR-6928
S2	**5.4E-4**	**Small LOCA** – *less than 2-inch diameter break*	NUREG/CR-6928

2.2.2.3 FEED LINE BREAK INSIDE CONTAINMENT

The feed line lengths listed in EPRI Table 6b are based on a 1-inch equivalent diameter pipe or greater, but not greater than 16 inches.

Table 9: Feed Line Break Frequencies				
Feed Line Break Inside Containment				
Break size	**U1 (ft.)**	**U1 Freq.**	**U2 (ft.)**	**U2 Freq.**
> 1 inch	248[11]	**4.9E-05**	230[12]	**4.5E-05**
3" to 8"	248	**1.7E-05**	230	**1.6E-05**
8" to 16"	248	**8.1E-06**	230	**7.5E-06**

2.2.3 BOUNDING THERMAL TRANSIENT EVENT FREQUENCY

The following assumptions are based on the results and insights from sections 2.2.2, Thermal Initiating Event Frequency, and 2.2.4, Mitigating Systems.

---

[10] *The pipe lengths are the same because a big pipe can have all of the different break sizes listed in the table. For example you can have a 1 inch hole in an 18 inch pipe or a 3 inch hole in an 18 inch pipe. The length of 18 inch pipe is the same for all of the break sizes.*

[11] *Piping Isometric Dwg. P-112*

[12] *Piping Isometric Dwg. P-212*

Assumptions:

1. ***LOCA (>2 inch), SLBI (>3inch), and FLBI (>3inch)*** are conservatively assumed to provide a 270 °F or greater challenge to the CT. For feedwater line breaks, the total energy released to the containment is lower because much of the feedwater flows directly into containment without being boiled in the SG.
2. **Breaks smaller than those listed in assumption 1 are not included**. Small LOCA (< 2-inch) is a minor challenge to containment temperature within capacity of the normal containment cooling system. These breaks are assumed to approach the capacity of the normal containment cooling system and present a negligible challenge. Although frequency of the smaller break events is higher, only a containment cooling mitigation failure will result in a temperature that will challenge the CT design. Section 2.2.4, Mitigating Systems, shows that the probability of having all containment cooling systems available is very high, ~1.0. The capability of the mitigating functions minimizing the consequences of the DBA 1.4ft^2 break, MSLB, is summarized in Table 10. These results show that systems that control containment temperature have a significant impact on peak containment temperatures. A sensitivity that considers the impact of smaller breaks is provided in Section 2.2.6.

From Table 7: For SLBI greater than 3 inch, the frequency is 1.4E-4 for Unit 1 and 1.9E-4 for Unit 2.
From Table 8: LOCA> 2 inch = 5.1E-4 for both units.
From Table 9: For FLBI greater than 3 inch, the frequency is 2.5E-5 for Unit 1 and 2.4E-5 for Unit 2.

The total demand frequency is:

LOCA (> 2 inch) + SLBI (> 3 inch) + FLBI (>3 inch) = CT Thermal Transient Initiator Frequency

Unit 1 = 5.1E-4 + 1.4E-4 + 2.5E-5 = **6.8E-4/year**
Unit 2 = 5.1E-4 + 1.9E-4 + 2.4E-5 = **7.3E-4/year**

The higher of the two initiating event frequencies, **7.3E-4**, will be used to quantify △CDF.

2.2.4 MITIGATING SYSTEMS

This section examines failures of systems that mitigate containment temperature transients. Insights from this review will determine what, if any, system failures must be considered in the hazard frequency calculation.

The design basis analysis considered worst case single failures such as loss of 1 of 2 containment safeguards signals. Containment safeguards pressure signals automatically actuate containment fan coolers (CFC) and containment spray (CS) systems. Both these systems support containment heat removal.

The following table provides design basis GOTHIC model results for a 1.4 ft^2 break, ~16 inch, for various mitigating system configurations. The probability of each configuration is estimated based on the fault trees in Attachment A. Case 1 is the most likely, cases 2 through 5 consider a failure of one or more functions credited in case 1.

Table 10: 1.4 ft² DBA Temperatures Based on 100% Power – MSLB							
Case	**ACT**	**CFC**	**CS**	**FIV**	**Containment Temp[13] (F)**	***Configuration Probability[14]***	**Temperature Basis**
1	2 of 2 *(No Failures)*	4 of 4 *(No Failures)*	2 of 2 *(No Failures)*	SUCCESS	<270 *estimated*	~1	*Assumption based on results from Case 2. All trains available should result in temperatures less than 270F.*
2	1 of 2 *(One Train Fails)*	2 of 4 *(One Train Fails)*	1 of 2 *(One Train Fails)*	SUCCESS	270	<1E-5	*CN-CRA-08-43 Rev 01 Case 1c, Table 5-1*
3	2 of 2 *(No Failures)*	4 of 4 *(No Failures)*	2 of 2 *(No Failures)*	FAILURE	277.8	~1E-3	*CN-CRA-08-43 Rev 01 Case 1b, Table 5-1*
4	1 of 2 *(One Train Fails)*	2 of 4 *(One Train Fails)*	None *(Both Trains Fail)*	SUCCESS	>280	<1E-5	*Assumption, total frequency <1E-05 and as such can be screened out*
5	1 of 2 *(One Train Fails)*	1 of 4 *(One Train Fails + 1)*	1 of 2 *(One Train Fails)*	SUCCESS	>280	<1E-5	*Assumption, total frequency <1E-05 and as such can be screened out*

Probability *= probability of containment safeguards configuration*
ACT *= Containment Safeguards Actuation Trains Available*
CFC *= Containment Fan Coolers (4 coolers total, 2 per train) Available*
CS *= Containment Spray (2 pump trains) Available*
FIV *= Feedwater Isolation Valve (failure to isolate primary valve allowing water upstream to inject)*

Minimum success per FSAR: 1 of 2 ACT, 2 of 4 CFC, 1 of 2 CS OR 1 of 2 ACT, 4 of 4 CFC OR 1 of 2 ACT, 2 of 2 CS

The referenced tables and figure from CN-CRA-08-43 Rev 01:

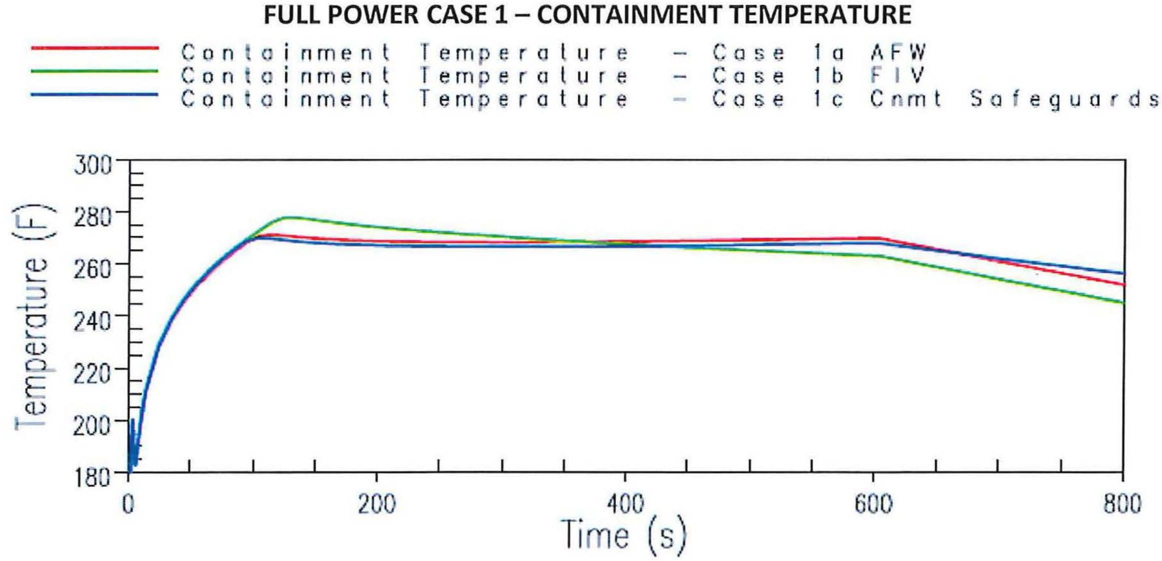
Table 4-1

SLB Inside Containment Case Definitions			
	Single Failure		
Initial Power	Auxiliary Feedwater Runout Protection	Feedwater Isolation Valve	Containment Safeguards
Full	1a	1b	1c

Table 5-1

Containment Response System Actuation Timing and Peak Pressure and Temperature Results						
	Fan Cooler [(1)]		Cont. Sprays		Peak Pressure (psig @ sec)	Peak Temperature (°F @ sec)
Case	Hi-1 Setpoint Reached (sec)	Fan Coolers Start (sec)	Hi-2 Setpoint Reached (sec)	Cont. Sprays Start (sec)		
1a	1.1	71.2	36.6	92.6	48.77 @ 114.2	271.4 @ 114.2
1b	1.1	71.2	36.0	92.0	53.62 @ 134.2	277.8 @ 124.2
1c	1.1	71.2	36.3	92.3	47.77 @ 104.2	270.1 @ 104.2

FULL POWER CASE 1 – CONTAINMENT TEMPERATURE



[13] *Containment is model as a single bulk volume as such the temperature is the bulk average temperature.*
[14] *Fault trees were used to calculate the probability of the temperature end states – refer to Attachment A.*

The following table summarizes the hazards based on the data from Table 10 using the Unit 2 "IE Frequency" from section 2.2. Unit 2 frequency is conservative since it is slightly greater than Unit 1.

Table 13: Thermal Hazard *[based on Mitigating System Availability]*					
IE Frequency	**Mitigation Failure**	**Mitigation Probability**	**Total Frequency**	**Temp (F)**	**Basis**
7.3E-4	None	1	7.3E-4	270	Conservatively assume 3-inch break will reach 270 F
7.3E-4	1 Train	~1E-3	7.3E-7	278	
7.3E-4	1+ Trains	~1E-5	7.3E-9	N/A	Low frequency cases neglected – can assume CT failure

As shown above, the first point on the hazard curve, 7.3E-4, dominates as additional failures leading up to a single train of safeguards or FIV failure has a minor impact on temperature with a relatively low probability.

In summary, the following failures and initiating events were considered in developing this hazard curve, but shown not to be governing:

- **AFW Failure** – failure of AFW to actuate and inject puts less mass (and therefore energy as steam) into the containment.
- **FIV failure** – failure of the primary isolation valve is analyzed to account for the water in the piping until a redundant isolation valve closes. Failure of both valves is low probability and can be neglected.
- **MSIV failure** – the failure probability of a MSIV and non-return valve allowing blowdown of both steam generators is low and can be neglected.
- **Pressure Induced Tube(s) Rupture** – not assumed likely or a significantly different challenge.

In conclusion, the thermal hazard frequency is bounded by the Unit 2 frequency of 7.30E-04/year.

2.2.5 THERMAL CONVOLUTION CALCULATIONS

When the conservative assumptions developed in the previous sections are considered along with a bounding CCDP of 1.0, the CDF is assessed to be below 1E-6/year:

Frequency of Temp 270-278 (Hazard Frequency) * Probability of CT Failure (Fragility)

Table 14: Thermal Convolution Results		
Hazard Frequency	**Fragility (286F)**	**Failure Frequency**
7.30E-04/year	**< 0.001**	**< 7.30E-07/year**

Since there is no thermal design basis for the CT the total CDF is considered the Δ **CDF;** as such **ΔCDF = 7.3E-07.** This value meets the RG 1.174 criterion of <1E-6.

2.2.6 THERMAL SENSITIVITY ANALYSIS

Assume all steam and water line breaks inside containment result in temperatures that overstress the CT. Then The total demand frequency is:

Table 15a: Feed Line Break Frequencies				
Feed Line Break Inside Containment				
Break size	**U1 (ft)**	U1 Freq.	**U2 (ft)**	U2 Freq.
1" to 3"	248	**4.90E-05**	230	**4.50E-05**
3" to 8"	248	**1.70E-05**	230	**1.60E-05**
8" to 16"	248	**8.10E-06**	230	**7.50E-06**
	TOTAL	**7.41E-05**		**6.85E-05**

Table 15b: Steam Break Frequencies				
Steam Line Break Inside Containment				
Break size	**U1 (ft)**	**U1 Freq.**	**U2 (ft)**	**U2 Freq.**
1" to 3"	215	**2.00E-04**	294	**2.80E-04**
3" to 8"	215	**5.60E-05**	294	**7.70E-05**
8" to 16"	215	**5.20E-05**	294	**7.10E-05**
16" to 30"	215	**3.50E-05**	294	**4.30E-05**
	TOTALS	**3.43E-04**		**4.71E-04**

Table 16: LOCA Frequencies		
PRA Designator	**Frequency** *per year*	**Description**
A	**1.3E-6**	**Large LOCA** – *greater than 6-inch diameter break*
S1	**5.1E-4**	**Medium LOCA** - *2 to 6-inch diameter break*
S2	**5.4E-4**	**Small LOCA** – *less than 2-inch diameter break*
TOTAL	**1.05E-03**	

ALL LOCA + ALL STEAM + ALL Feed Line = CT Thermal Transient Initiator Frequency

Unit 1 = **1.05E-03**+ **3.43E-04**+ **7.41E-05** = **1.47E-03/year**
Unit 2 = **1.05E-03**+ **4.71E-04**+ **6.85E-05** = **1.59E-03/year**

Assuming all line breaks result in temperatures that challenge CT then the frequency = 1.59E-06 versus the 7.30E-07 assuming LOCAs > 2 inch, SLBIs > 3 inch), and FLBIs >3 inch. As noted previously, the smaller line break event and FLBIs are not expected to result in temperatures that overstress the CT.

Table 17: Thermal Convolution Results – All Line Breaks		
Hazard Frequency	**Fragility (286F)**	**Failure Frequency**
1.59E-03/year	**< 0.001**	**< 1.59E-06/year**

3.0 TOTAL RISK – BOUNDING CASE

For the bounding case, the total △ CDF is calculated as follows:

$$Seismic\ \triangle CDF + Thermal\ \triangle CDF = Total\ \triangle CDF$$

CASE	Total △CDF	Assumptions
Base	1.88E-06 + 7.30E-07 = **2.60E-06**	*Assumed small LOCA or SLB1 between 1 inch and 3 inch did not result in a significant temperature transient.*
Sensitivity	1.88E-06 + 1.59E-06 = **3.46E-06**	*Assumed 1 inch and greater steam and waterline breaks inside containment result in a thermal transient overstressing CT*

Including small line breaks and FLBIs increases **△CDF** for the bounding case by 33%. The analysis of the CT for the applied thermal loads is included in Reference 12 and the CT is shown to maintain structural integrity and not affect the functions of the attached or adjacent SSCs. The results of this sensitivity does not change the conclusions of this evaluation; RG 1.174 Region II criterion of <1E-5 are met.

Qualitative factors that reduce risk are discussed in the next section.

4.0 QUALITATIVE FACTORS

There are several conservative and significant considerations not included in the bounding analysis. These factors suggest there is significant risk margin beyond the value quantified assuming a CCDP=1.0.

4.1 SEISMIC LOADING DIRECTION.

Seismic calculations, references 12c and 12d, note the following:

Upon inspection of the layout of the dome truss, a critical direction for the load case without liner contact is taken to be acceleration in the direction 10° east of North. Applying the seismic loading in this direction results in the greatest load in the T2 bottom chords.

Since the maximum loading direction is related to the random nature of the seismic event, it is likely that the CT structure will not experience seismic loading in the critical direction; i.e. loads vary with seismic load direction. Although detailed structural calculations have not been performed for other loading directions, it is reasonable to assume that the capacity of the CT in other loading directions would be greater than the critical direction of 10°.Expected risk reduction 10%-50%.

4.2 "All Design Basis" △CDF based on Modifications that Exceed Design Requirements

Since it is not possible to design a modification that exactly meets design basis requirements, the design compliant A_m reflects a CT with modifications that exceed design requirements. This results in a smaller CDF for the "all design basis mod" case and consequently a higher △CDF. Expected risk reduction less than 10%

4.3 CT Over Stress will NOT Always Lead to Risk Significant Component Failure

Reference 3 assessed the impact that falling CT debris would have on components below and concluded that many of the critical components within containment are robust, e.g. Main Steam lines, PORVs, etc.; and many are protected by robust barriers, the SI system for example. As such it is reasonable to assume these SSCs will survive and operators will have mitigating options available should the CT fail. Expected risk reduction more than 90%.

4.4 CT Over Stress will NOT Always Lead to a CT Collapse

When overstressed, the CT structure will likely remain intact or not fully detach from its supports. This is most likely at the lower intensity seismic initiating events. The structural analyses evaluated the probability of overstress and did not assess the stability of the CT structure and its components, i.e. the probability that the CT would deconstruct fully or partially. Page 26 of the Eng. Evaluation [Ref 6] notes the PGA for the GMRS based earthquake as 0.14g, therefore it is reasonable to assume that at 0.34g the CT remains intact and stable although slightly overstressed. Applying this assumption to the bounding analysis would reduce its △CDF from 2.69E-6 to **1.28E-06**; from 9.38E-08 to **3.08E-8** for the demonstrably conservative case convolved with IPEEE. Expected risk reduction is >50%.

5.0 DEMONSTRABLY CONSERVATIVE ANALYSIS

To assess the level of conservatism of the bounding analysis, a "demonstrably conservative analysis" was developed using a limited scope PRA. Demonstrably conservative analysis uses assumptions such that the assessed outcome will be conservative relative to the expected outcome [Ref 10].

This analysis applied more realistic assumptions, assuming it is unlikely that a CT collapse will always lead to core damage; applying a more realistic assumption that CCDP is less than 1.0. However, the conservative assumption that the CT will collapse if overstressed was retained.

For this assessment the following aspects of a CT failure and its consequences were studied:

- How the CT fails when overstressed
- Trajectory of components from the failed CT
- Vulnerability of risk significant components to falling CT debris
- Location and robustness of the barriers that would protect critical components from the falling CT debris.

The results of this assessment [Ref 3] identified SSCs that are likely to survive a CT failure and operator actions that remain viable under seismic or thermal (e.g. LOCA) transients. Although this analysis is more realistic than the bounding analysis, conservative assumptions were applied to assure key uncertainties are addressed or bounded, for example:

- The CT structure is always assumed to immediately generate falling debris when overstressed.
- CT debris targeting critical SSCs are assumed to be oriented in a way that maximizes damage to targeted SSCs.
- Performance shaping factors, factors affecting operator actions, were increased to address additional stress and concurrent critical actions that would reduce the reliability of operator actions.

The CT is assumed to deconstruct if overstressed. Note that the structural calculations, Ref 6 and 12, did not include an assessment of the consequences of overstress, i.e. these calculations did not assess CT failure mechanisms related to CT connections or structural member failures that would result in deconstruction or collapse of the CT or its components. The CT consists of trusses that are welded and bolted to cross members. This assessment makes a general assumption that bolted connections will fail before welded connections; that the trusses will fall intact and bolted sections will separate and fall. The largest truss weighs ~3 tons and is assumed oriented in a way that maximizes damage. Cross members are estimated to weigh ~500lbs. These assumptions bound the impact that falling CT debris, trusses and cross members, will have on objects below. More details are provided in the qualitative evaluation of the impact of trusses and cross members provided in the Point Beach CT Target Assessment, reference 3.

Many of the potential targets below the CT are SSCs that are fairly robust, e.g. Main Steam lines, PORVs, etc.; and many are protected by robust barriers, the SI system for example. As such it is reasonable to assume there is some probability that operators will have mitigating options available should the CT failure damage SSCs below.

5.1 TARGET ASSESSMENT INSIGHTS

A detailed evaluation of the postulated failure of an overstressed CT and its consequences is documented in Reference 3. The following summarizes the insights from the Target Notebook for targets above EL 66 (penetration of EL 66 floor was determined to be unlikely):

1. **MAIN STEAM VENT LINES** are small (1 ½ inch) and can be ruptured by very heavy falling debris such as an intact truss. Although most of the piping is close to the containment wall, away from where most debris would fall, the probability of a small steam line break inside containment (SLBI) is assumed to be high. These main steam vent lines (penetrations P57 and P58 are at El 88) are not a large early release issue because this piping is normally isolated during power operation (there are normally closed valves outside containment).
2. **MAIN STEAM LINE PIPES**. The main steam piping wall thickness is ~1 inch. Only an intact truss oriented in a way to minimize contact area (i.e. maximize energy transfer) and targeted directly on the centerline of the pipe may damage the pipe. Therefore penetration/puncture is assumed to be very unlikely.
3. **POLAR CRANE** is considered rugged and unlikely to fail in a gross way due to falling CT debris. The polar crane main hook capacity is 100 tons as compared to the largest truss which weighs ~3 tons [Ref 3].
4. **CONTAINMENT SPRAY PIPING** is attached to the CT and is assumed to fail. Containment spray is not credited in the PRA model of record.
5. **CONTAINMENT WALL** is considered very rugged and will not fail in a gross way, i.e. result in a large early release. A containment hole size of 2 inches in diameter is the threshold for a large release; a hole size less than or equal to a 2 inch diameter will not result in a large release [WCAP-15791-P, Rev. 1]
6. **CONTAINMENT PENETRATIONS** with isolations outside containment were evaluated and are considered reliable outside containment. The main steam penetrations and feedwater penetrations that are sealed inside containment are rugged and located and configured in a way that they are not vulnerable.
7. **CONTAINMENT VENT PURGE SUPPLY AND EXHAUST** (penetrations V-1 and V-2 are at El 98) – this system is not normally operating during power operation; there is a normally closed valve outside and the inside is blank flanged.
8. **CONTAINMENT SAMPLE LINES** (penetrations X-1 and X-2 in the C-2 personnel lock at El 69), but these 1-inch lines have fail closed air operated valves outside containment.
9. **DECK PLATE LOCATED at the EL 66 ABOVE the SEAL TABLE**. The deck plate has minimal resistance to a falling truss oriented in a way that maximizes damage. There is some structural steel located above the seal table. These serve to provide some protection to the tubing below that could result in a small break LOCA if damaged.
10. **PRESSURIZER PORVS** are at about El 76 on top of the pressurizer and under concrete missile shields designed to keep their valve stems from being ejected upward into the containment liner. The missile shield is 15-inch thick reinforced concrete and completely covers the top of pressurizer cubicle. This barrier is judged to provide adequate protection. The instrument air system which normally supplies the PORVs with motive force is not a seismic category 1 system and may not be available in the event of a large earthquake. EC284214 (Unit 2) and EC285145 (Unit 1) [Ref 13] are installing seismic category 1 nitrogen tanks and piping to supply the PORVs in the event of a loss of instrument air to the PORVs. These modifications are being done as part of the transition to NFPA 805 and will also reduce the consequence of a CT failure. There will be one tank for each PORV with an adequate supply of compressed gas to provide for 24-hour operation of each PORV. The tanks will be located on the 46 ft. elevation in

containment with associated piping routed on the 46 ft. elevation or in the pressurizer cubicle[15]. The associated cables, tubing and piping will be protected from falling debris. Regulators, tubing, and control valves are all located in the pressurizer cubicle or below the EL 66. The PORV solenoid valves are inside the pressurizer cubicle with the PORVs. Consequently the location of the planned modification to install nitrogen back-up supply to the PORVs will enhance protection from any failure of CTs.

11. **FEEDWATER LINES, REACTOR COOLANT PRESSURE BOUNDARY and SAFETY INJECTION PIPING** are below EL 66.

5.2 CCDP GIVEN CT FAILURE

Figures 4, 5 and 6 provide simplified event trees that show the sequences considered along with success and failure probabilities of critical systems and components. These event trees were developed assuming a seismic or thermal event caused a CT failure. Non-seismic SSCs like air compressors and instrument air lines are considered failed and not included. Seismic SSCs outside containment are assumed not to fail since they are designed for the design basis earthquake (DBE) and will not fail due to thermal transients inside containment.

The event tree and their top events are described in more detail as follows:

SEISMIC EVENT *FIGURE 4*	**STEAM LINE BREAK** *FIGURE 5*	**LOCA** *FIGURE 6*

The initiating events evaluated are a seismic event, steam line break, LOCA and feedwater line break. A separate event tree was developed for each initiating event. The event tree end state probabilities are developed assuming the entry initiating event is equal to 1.0. The seismic event tree is assumed to apply to the Feedwater line break event.

NO SLB

"NO SLB" success is when the CT failure does not break the main steam piping. Given susceptibility of the small 1 ½ inch main steam vent lines, a probability of failing of at least one SG vent line is assumed to be 0.9. Failure at this event is also assumed to fail half of the AFW capability.

NO SLB2

This node represents a main steam vent break to the 2nd SG given that the CT already caused a break at one SG. This would require multiple members to fail and drop on both sides of the Polar Crane. Given one steam line has been hit it is likely that the other will as well. Therefore a 0.9 probability is assumed for the 2nd steam line break. Failure at this event results in unavailability of all AFW (set as a Guaranteed Failure in the event tree).

AFW

Most AFW piping to the SGs is below EL 66. The probability of failing lines to both SGs given no impact to the steam lines is judged to be low (0.01). The probability of failing the line to one available SG is also small (0.1). Failure of AFW requires F&B as success in the event tree. Note that these failure probabilities are assumed to subsume independent failure probabilities of the AFW system. Seismic failure of the AFW system should not be limiting based on review of IPEEE. An earthquake large enough to fail AFW would

[15] *Tubing from the fixed gas bottles is routed so that it penetrates the pressurizer cubicle wall on the 46' elevation. This ensures that all PORV pneumatic backup tubing outside of the pressurizer cubicle is below EL 66 ft. This reduces the risk of a loss of feed and bleed function on a postulated failure-of the CT.*

likely result in core damage regardless of CT failure based on the way the event tree is built. This would not credit F&B for decay heat removal if all AFW is lost. The planned NFPA 805 mods [Ref 11] will reduce the total CDF by protecting the F&B SSCs above EL 66. Typically seismic induced support system failures that would fail AFW would lead to core damage.

NO LOCA

The probability that debris from the CT has the following impacts is included for completeness even though the analysis indicates this is unlikely: the seal table, pressurizer cubicle, and fails piping or safety valve, PORV path on the pressurizer causing a LOCA. A 0.1 probability is conservatively assumed. Instead of guaranteeing success at the next top event, the model conservatively assumes a very small LOCA requiring a PORV to succeed at top event **PORVs** even when AFW is successful (i.e., F&B is assumed to be required).

PORVS

The probability that the CT impacts the pressurizer cubicle and fails PORVs so they cannot open is unlikely based on the evaluation in Section 5.1.10 of the CT Target Assessment [Ref 3]. The model conservatively assumes a very small LOCA rather than assume the LOCA is large enough to guarantee bleed (equivalent to PORV success) even when AFW is successful. F&B unreliability due to operators failing to initiate F&B or PORVs failing to open is judged to dominate this top event failure:

- PORVs = 2.6E-3 (failure PORV and Block valve path – assumes both paths are required)
- HEP = 5E-2 (baseline value for internal events PRA – detailed evaluation provided in Attachment B)

SI

Safety injection piping is below EL 66 where it is unlikely to be impacted. Conservatively a 0.01 probability is assigned to this node. This probability subsumes independent failures associated with unavailability of SI and sump recirculation. Similar to AFW, seismic failure of the SI system is not limiting based on review of IPEEE; an earthquake large enough to fail SI would likely result in core damage regardless of CT failure. Typically seismic induced support system failures that fail SI would cause core damage.

CD or OK

The end state of the sequence is core damage, "CD", or "OK". The "OK" indicates that that CT failure did not cause core damage. The CD value is the probability that core damage is due to a failure of the CT, i.e. conditional core damage probability (CCDP) due to a CT failure. The event tree for the most part neglects other independent failures, which are judged to be either small contributors or other seismic CDF contributors independent of CT failure.

5.2.1 SEISMIC EVENT CCDP

The following event tree illustrates the key sequences related to a seismic event that overstresses the CT sufficiently to result in some level of disassembly; i.e. with debris falling and potentially striking key components and affecting functions represented by the top nodes.

Sequence 1: The sequence results in a successful mitigation of core damage due to a CT failure. No steam line break or LOCA occurs. AFW and PORVs are available and SI is successful.

Sequence 17: Core damage occurs as a result of two concurrent steam line breaks which fail AFW, PORVS fail and without a bleed facilitated by a LOCA feed is inadequate, i.e. SI fails.

The other sequences are variations of these two sequences.

Figure 4: CCDP ESTIMATE FOR CT FAILURE DUE TO SEISMIC EVENT

SEISMIC EVENT	No SLB	No SLB2	AFW	No LOCA	PORVs	SI	Seq	CD or OK
	0.1		0.99	0.9			1	OK
				0.1	0.95	0.99	2	OK
						0.01	3	1.0E-04
					0.05		4	5.0E-04
			0.01	NA	0.95	0.99	5	OK
						0.01	6	1.0E-05
					0.05		7	5.0E-05
	0.9	0.1	0.9	0.9			8	OK
				0.1	0.95	0.99	9	OK
						0.01	10	9.0E-05
					0.05		11	4.5E-04
			0.1	NA	0.95	0.99	12	OK
						0.01	13	9.0E-05
					0.05		14	4.5E-04
		0.9	*Guaranteed Failure*	NA	0.95	0.99	15	OK
						0.01	16	8.1E-03
					0.05		17	4.1E-02
						TOTAL CCDP =		**5.0E-02**

The following table summarizes the △**CDF** resulting from applying a seismic CCDP of 5.0E-02 to the bounding case.

SEISMIC RESULTS SUMMARY – DEMONSTRABLY CONSERVATIVE Analysis		
MODIFICATIONS	**CDF**	**ΔCDF,** *CCDP = 0.05*
Thermal Mod Unit 1, Unit 2 Unmodified	6.30E-06 * 0.05 = **3.15E-07**	**1.34E-07**
All Design Basis Mods (Base CDF)	3.61E-06 * 0.05 = **1.80E-07**	

The following table convolves the IPEEE seismic results with the ΔCDF results of the previous table using the method described in section 2.1.4.

CT U1 Mod, U2 As is [CASE 1]					IPEEE [CASE 3]		CASE1 + CASE3 - (CASE1*CASE3)	
	Hazard							
ID	**Description**	**Hazard Frequency**	**Probability of Failure X 0.05**	**Frequency**	**Probability of Failure**	**Frequency**	**Probability of Failure**	**Frequency**
%G01	Seismic Initiator (0.05g to <0.12g)	3.04E-04	5.98E-07	1.82E-10	4.64E-05	1.41E-08	4.70E-05	1.43E-08
%G02	Seismic Initiator (0.12g to <0.23g)	4.82E-05	5.10E-04	2.46E-08	1.34E-02	6.45E-07	1.39E-02	6.69E-07
%G03	Seismic Initiator (0.23g to <0.34g)	1.01E-05	7.73E-03	7.80E-08	1.45E-01	1.47E-06	1.52E-01	1.53E-06
%G04	Seismic Initiator (0.34g to <0.45g)	3.36E-06	2.15E-02	7.22E-08	3.78E-01	1.27E-06	3.91E-01	1.31E-06
%G05	Seismic Initiator (0.45g to <0.56g)	1.54E-06	3.36E-02	5.18E-08	5.96E-01	9.18E-07	6.10E-01	9.39E-07
%G06	Seismic Initiator (0.56g to <0.67g)	7.72E-07	4.14E-02	3.19E-08	7.53E-01	5.82E-07	7.64E-01	5.89E-07
%G07	Seismic Initiator (0.67g to <0.78g)	4.34E-07	4.56E-02	1.98E-08	8.54E-01	3.71E-07	8.61E-01	3.73E-07
%G08	Seismic Initiator (0.78g to <0.89g)	2.56E-07	4.78E-02	1.22E-08	9.14E-01	2.34E-07	9.19E-01	2.35E-07
%G09	Seismic Initiator (0.89g to <1g)	1.54E-07	4.89E-02	7.53E-09	9.50E-01	1.46E-07	9.52E-01	1.47E-07
%G10	Seismic Initiator (>1g)	3.36E-07	4.96E-02	1.67E-08	9.76E-01	3.28E-07	9.78E-01	3.28E-07
			Total CDF	**3.15E-07**		**5.97E-06**		**6.14E-06**

CT All Design Basis Mods [CASE 2]							CASE2 + CASE3 - (CASE2*CASE3)	
%G01	Seismic Initiator (0.05g to <0.12g)	3.04E-04	3.84E-08	1.17E-11	4.64E-05	1.41E-08	4.64E-05	1.41E-08
%G02	Seismic Initiator (0.12g to <0.23g)	4.82E-05	9.31E-05	4.49E-09	1.34E-02	6.45E-07	1.35E-02	6.50E-07
%G03	Seismic Initiator (0.23g to <0.34g)	1.01E-05	2.75E-03	2.77E-08	1.45E-01	1.47E-06	1.47E-01	1.49E-06
%G04	Seismic Initiator (0.34g to <0.45g)	3.36E-06	1.12E-02	3.76E-08	3.78E-01	1.27E-06	3.85E-01	1.29E-06
%G05	Seismic Initiator (0.45g to <0.56g)	1.54E-06	2.23E-02	3.43E-08	5.96E-01	9.18E-07	6.05E-01	9.32E-07
%G06	Seismic Initiator (0.56g to <0.67g)	7.72E-07	3.21E-02	2.48E-08	7.53E-01	5.82E-07	7.61E-01	5.88E-07
%G07	Seismic Initiator (0.67g to <0.78g)	4.34E-07	3.91E-02	1.70E-08	8.54E-01	3.71E-07	8.60E-01	3.73E-07
%G08	Seismic Initiator (0.78g to <0.89g)	2.56E-07	4.35E-02	1.11E-08	9.14E-01	2.34E-07	9.18E-01	2.35E-07
%G09	Seismic Initiator (0.89g to <1g)	1.54E-07	4.63E-02	7.12E-09	9.50E-01	1.46E-07	9.52E-01	1.47E-07
%G10	Seismic Initiator (>1g)	3.36E-07	4.83E-02	1.62E-08	9.76E-01	3.28E-07	9.78E-01	3.28E-07
			Total CDF	**1.80E-07**		**5.97E-06**		**6.05E-06**

ΔCDF =	**9.38E-08**

The following table summarizes the results of the convolution of the demonstrably conservative CDF and the IPEEE CDF.

SEISMIC RESULTS SUMMARY – DEMONSTRABLY CONSERVATIVE Analysis Convolved with IPEEE Seismic		
MODIFICATIONS	**CDF**	**ΔCDF,** *CCDP = 0.05*
Thermal Mod Unit 1, Unit 2 Unmodified	**6.14E-06**	**9.38E-08**
All Design Basis Mods (Base CDF)	**6.05E-06**	

5.2.2 STEAM LINE BREAK EVENT CCDP

This event tree assumes the first steam line break is a guaranteed failure, i.e. the steam line break is the initiating event. The balance of the nodes and probabilities are the same as the seismic event tree.

The sequences associated with "No SLB" are eliminated since these all have a 0.0 probability. The remaining sequences are the same as those in the seismic event tree but with a slightly higher probability of 5.5E-02.

Figure 5: THERMAL EVENT TREE FOR STEAM LINE BREAK

STEAM LINE BREAK	No SLB	No SLB2	AFW	No LOCA	PORVs	SI	Seq	CD or OK
	0		0.99	0.9			1	OK
				0.1	0.95	0.99	2	OK
						0.01	3	0.0E+00
					0.05		4	0.0E+00
			0.01	NA	0.95	0.99	5	OK
						0.01	6	0.0E+00
					0.05		7	0.0E+00
	1.0	0.1	0.9	0.9			8	OK
				0.1	0.95	0.99	9	OK
						0.01	10	1.0E-04
					0.05		11	5.0E-04
			0.1	NA	0.95	0.99	12	OK
						0.01	13	1.0E-04
					0.05		14	5.0E-04
		0.9	*Guaranteed Failure*	NA	0.95	0.99	15	OK
						0.01	16	9.0E-03
					0.05		17	4.5E-02
						TOTAL CCDP =		5.5E-02

Hazard Frequency x Fragility x CCDP = Failure Frequency

STEAM LINE BREAK			
Steam Line Break Frequency	**Fragility (275F)**	**CCDP**	**CDF**
1.4E-04/year	< 0.001	5.5E-02	< 7.7E-09

5.2.3 LOCA EVENT CCDP

This event tree assumes a small LOCA initiator. "NO LOCA" success is set to 0.0 and failure is set to 1.0.

The sequences are the same as those in the seismic event tree but with different probabilities.

Figure 6: THERMAL EVENT TREE FOR A LOCA INITIATING EVENT

LOCA	No SLB	No SLB2	AFW	No LOCA	PORVs	SI	Seq	CD or OK
	0.1		0.99	0			1	OK
				1	0.95	0.99	2	OK
						0.01	3	1.0E-03
					0.05		4	5.0E-03
			0.01	NA	0.95	0.99	5	OK
						0.01	6	1.0E-05
					0.05		7	5.0E-05
	0.9	0.1	0.99	0			8	OK
				1	0.95	0.99	9	OK
						0.01	10	9.0E-04
					0.05		11	4.5E-03
			0.1	NA	0.95	0.99	12	OK
						0.01	13	9.0E-05
					0.05		14	4.5E-04
		0.9	*Guaranteed Failure*	NA	0.95	0.99	15	OK
						0.01	16	8.1E-03
					0.05		17	4.1E-02
							TOTAL CCDP =	6.1E-02

The resulting CDF for a LOCA event is 3.12E-08:

LOCA			
LOCA Frequency	**Fragility (275F)**	**CCDP**	**CDF**
5.11E-04/year	< 0.001	6.1E-02	< 3.12E-08

5.2.4 FEEDWATER LINE BREAK EVENT CCDP

The feedwater line break event tree is identical to the seismic event tree; the initiating event frequencies are, of course, different. The following table provides the resulting CDF:

FEEDWATER LINE BREAK			
FEEDWATER LINE BREAK Frequency	Fragility (275F)	CCDP	CDF
2.5E-04/year	< 0.001	5.0E-02	< 1.25E-09

5.2.5 TOTAL THERMAL EVENT CDF

The total CDF is assumed to be the △CDF

	Line Break Frequency	Fragility (275F)	CCDP	△CDF
STEAM LINE BREAK	1.4E-04/year	< 0.001	5.5E-02	7.70E-09
LOCA	5.11E-04/year	< 0.001	6.1E-02	3.12E-08
FEEDWATER LINE BREAK	2.5E-04/year	< 0.001	5.0E-02	1.25E-09
			TOTAL	4.02E-08

The total thermal event CDF meets RG 1.174 Region III criterion of <1E-6 △CDF.

5.3 TOTAL RESULTS - DEMONSTRABLY CONSERVATIVE ANALYSIS

RESULTS SUMMARY – DEMONSTRABLY CONSERVATIVE SEISMIC + THERMAL		
SEISMIC	THERMAL	△CDF
1.34E-07	4.02E-08	1.74E-07

RESULTS SUMMARY – DEMONSTRABLY CONSERVATIVE SEISMIC * IPEEE +THERMAL		
SEISMIC CONVOLVED with IPEEE SEISMIC	THERMAL	△CDF
9.38E-08	4.02E-08	1.34E-07

In both cases the total CDF meets RG 1.174 Region III criterion of <1E-6 △CDF

6.0 LERF REVIEW

For the cases presented in Section 1.7 a conditional large early release probability (CLERP[16]) of 0.2 was applied. The following considerations provide a qualitative basis for the validity of this assumption.

1. Point Beach CT Target Assessment, Reference 3, assessed CT failure impact on the internal containment structure and containment penetrations and showed that it is unlikely that containment and its penetrations will be damaged. Containment failure is unlikely due to the robust nature of the penetrations, their locations, redundant isolations outside containment, and barriers shielding them from falling debris. Moreover, there are only a limited number of penetrations that can be targeted by CT debris. This is based on conclusions in Reference 3 that are based on consideration of inputs provided during confirmatory containment walkdowns, a review of spatial interactions, target impact analysis, and CT target analysis. Reference 3 validates that it is conservative to assume that CLERP associated only with a CT failure is 0.2.

2. Regarding thermal transients, since Reference 3 showed that debris resulting from a CT failure is unlikely to damage SSCs that are important to LERF, the LERF from the CT failure events remains consistent with LERF from the internal events PRA model and therefore can be represented by the internal events CLERP. The table below provides the CLERP values for all Point Beach hazards; all of which are less than 0.2.

	CLERP	
HAZARD	Unit 1	Unit 2
Internal Events at Power	0.007	0.007
Internal Floods at Power	0.067	0.067
Internal Fire at Power	0.015	0.016
High Winds at Power	0.033	0.045
Seismic	0.194	0.194
Other Hazards	0.100	0.100
TOTAL	0.032	0.030

3. The sensitivity study below tabulates the resulting LERF values resulting from varying CLERP assumptions. The study shows that for all the demonstrably conservative cases, a CLERP of 1.0 would still meet Region II LERF criterion. A CLERP of 0.38 would still meet Region II criterion for ΔLERF for the bounding case.

---

[16] CLERP=LERF/CDF

The LERF results are color coded in accordance with the most restrictive RG 1.174 region the value complies with.

	ΔCDF	ΔLERF
REGION I	>1.0E-05	>1.0E-06
REGION II	>1.0E-06 ⇨ 1.0E-05	>1.0E-07 ⇨ 1.0E-06
REGION III	⇨ 1.0E-06	⇨ 1.0E-07

Maximum CLERP
< 0.2
0.2 ⇨<0.5
0.5 ⇨ <1.0
1.0

		CASE	ΔCDF	ΔLERF based on CLERP values from 0.1 through 0.3			Maximum CLERP
				0.1	**0.2**	0.3	Region II =<1E-06
SEISMIC	1	**Bounding** *[section 2.1]*	**2.69E-06**	2.69E-07	**5.38E-07**	8.07E-07	**0.37**
SEISMIC	1a	**Bounding CT X Seismic IPEEE** *[section 2.1.4]*	**1.88E-06**	1.88E-07	**3.76E-07**	5.64E-07	**0.53**
SEISMIC	2	**Demonstrably Conservative** *[section 5]*	**1.34E-07**	1.34E-08	**2.68E-08**	4.02E-08	**1.00**
SEISMIC	2a	**Demonstrably Conservative X Seismic IPEEE** *[section 5.2.1]*	**9.38E-08**	9.38E-09	**1.88E-08**	2.81E-08	**1.00**
THERMAL	3a	**Bounding** *[section 2.2]*	**7.30E-07**	7.30E-08	**1.46E-07**	2.19E-07	**1.00**
THERMAL	3b	**Bounding Sensitivity (all SLBs)** *[Section 2.2.6]*	**1.59E-06**	1.59E-07	**3.18E-07**	4.77E-07	**0.63**
THERMAL	4	**Demonstrably Conservative** *[section 5]*	**4.02E-08**	4.02E-09	**8.04E-09**	1.21E-08	**1.00**
TOTAL	**Bounding,** Cases 1 + 3a		**3.42E-06**	3.42E-07	**6.84E-07**	1.03E-06	**0.29**
TOTAL	**Bounding convolved with IPEEE,** Cases 1a + 3a		**2.61E-06**	2.61E-07	**5.22E-07**	7.83E-07	**0.38**
TOTAL	**Demonstrably Conservative,** Cases 2 + 4		**1.74E-07**	1.74E-08	**3.48E-08**	5.23E-08	**1.00**
TOTAL	**Demonstrably Conservative convolved with IPEEE,** Cases 2a + 4		**1.34E-07**	1.34E-08	**2.68E-08**	4.02E-08	**1.00**

7.0 SHUTDOWN RISK

THERMAL TRANSIENTS

The thermal transients were assumed to occur only during full power operations. The analysis bounds shutdown risk conditions:

- **CONTAINMENT BARRIER NOT INTACT**: This condition would be bounded by the analysis for full power operations due to the extremely low probability of occurrence of a thermal event when the containment barrier could be breached for maintenance. Further, station operation would be limited to Modes 5, 6 or defueled during the postulated condition which restricts RCS temperature to ≤ 200°F. The supporting engineering evaluations conclude the CTs or supported equipment remain structurally stable and capable of performing their design basis functions unimpeded.

- **MIDLOOP OPERATION**: The initiating event, LOCA, would be bounded by the full power operation LOCA event. No MSLB event could occur at this condition. For the lesser LOCA event, the same need for F&B protection applies. No additional vulnerable targets are presented for the Residual Heat Removal System while operating in the decay heat removal mode. Very little time is spent in mid-loop operation.

SEISMIC EVENTS

Seismic risk during a shutdown is minimized by the hazard exposure being roughly 4% to 8% of the full power operation exposure per year; assuming 1 to 2 month outage duration per 18 month fuel cycle.

8.0 PRA QUALITY

8.1 BOUNDING ANALYSIS

A bounding PRA is the principal basis for this assessment. It assumes that CT overstress due to a seismic or thermal event directly leads to core damage; CCDP = 1.0. As such the bounding analysis did not, nor did it need to, credit mitigating systems and operator actions, therefore a full scope PRA was not required. Only seismic and thermal hazard data and fragility data are used to calculate core damage and large early release probabilities.

The following ASME RA-Sa-2009 sections apply:

- Part 5, Requirements for Seismic Events At-Power PRA [Ref. 10]
- Part 6, section 6-2, Technical Requirements for Screening and Conservative Analysis [Ref. 10]

There are no Facts & Observations (F&Os) from previous Point Beach peer reviews that apply to these sections.

The following elements require an assessment against the ASME and RG 1.200 quality requirements:

- Seismic Hazard Curve
- Seismic Fragilities
- Thermal Hazard Curve
- Thermal Fragilities
- Structural Calculations
- Technical Requirements for Screening and Conservative Analysis

Compliance to applicable RG 1.200 and ASME RA-Sa-2009 supporting requirements is documented in Attachment C.

8.2 DEMONSTRABLY CONSERVATIVE ANALYSIS

The demonstrably conservative analysis applied simple PRA methods to perform sensitivity analyses that were based in part on the bounding analysis. Screening values were used for SSC failure data. This data was applied to simple event trees used in the demonstrably conservative analyses. The event trees were quantified using EXCEL but could have easily been evaluated by hand.

As such the very simplified treatments of basic PRA methods do not require a peer review or additional scrutiny beyond the technical reviews performed for this evaluation. However ASME Part 5 and Part 6 are applicable as they are to the bounding analysis and compliance is documented in Attachment C.

9.0 UNCERTAINTIES

All known uncertainties are addressed by the simple bounding case and sensitivity analyses applied in this evaluation, Section 2. The "demonstrably conservative analysis", Section 5, shows there is significant risk margin contained within the bounding case. Moreover, the qualitative factors in Section 4 further validate the significant risk margin inherent in the bounding case results. Also, the structural analyses include many conservative assumptions that show that there is sufficient reserve capacity to preclude a catastrophic CT failure even at high accelerations.

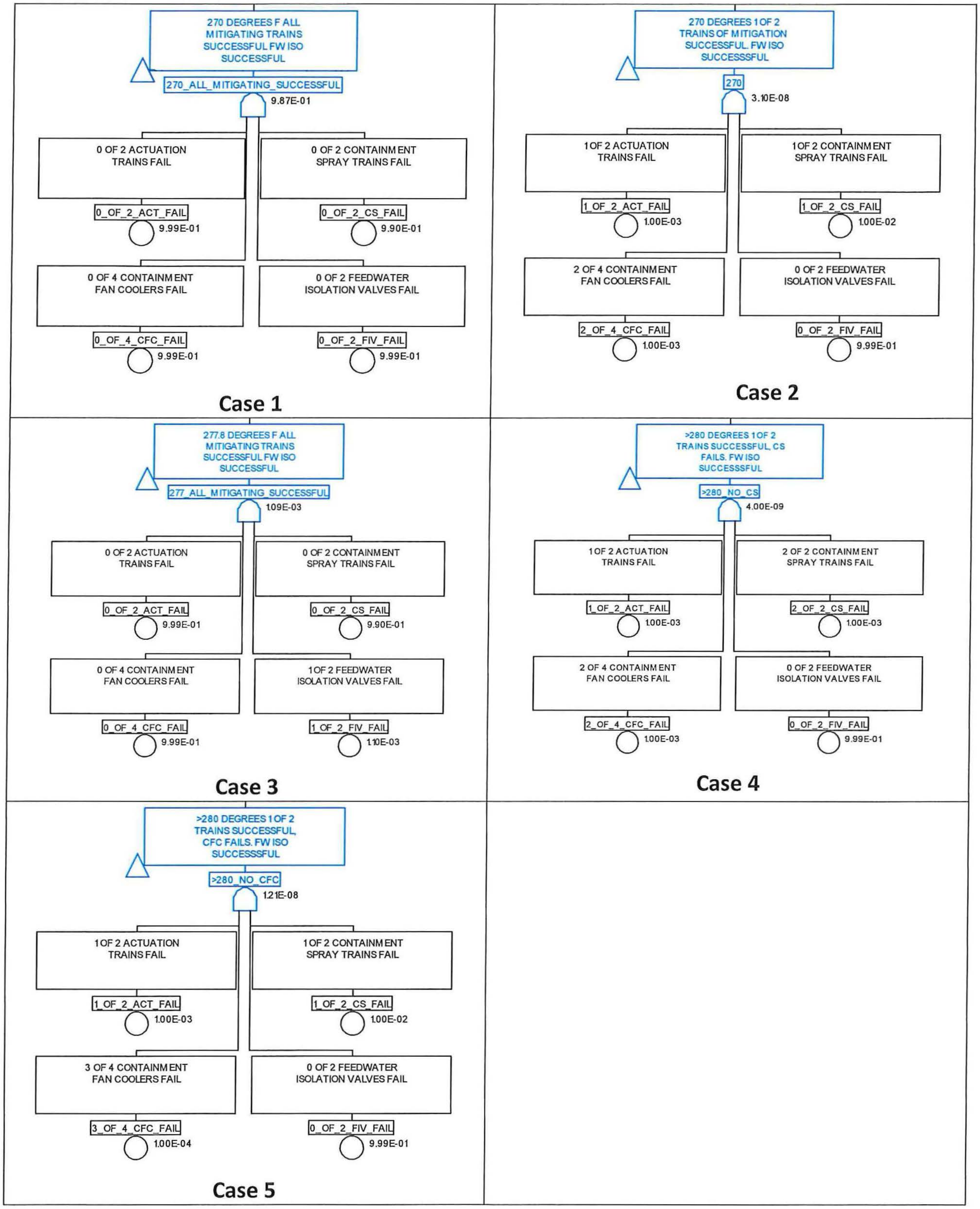
With all these factors considered, the application of the bounding case fully complies with RG 1.174 which states that a PRA should include a full understanding of the impacts of the uncertainties through either a formal quantitative analysis or a simple bounding or sensitivity analyses.

10.0 REFERENCES

1. CN-CRA-08-43 Rev 01,and Rev 01-A, "Point Beach EPU: Units 1 (WEP) and 2 (WIS) SLB Inside Containment Response
2. EPRI FRANX version 4.3
3. Point Beach CT Target Assessment, PBN-BFJR-17-020.
4. NRC 2014-0024 March 31, 2014, ML14090A275, NextEra Energy Point Beach. LLC Seismic Hazard and Screening Report (CEUS Sites) Response NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident
5. RSM-112013-037 Point Beach Hazard and Screening Report from EPRI
6. S&A calculation 11Q0060-C-028, Seismic Fragility Analysis of Containment Dome Trusses
7. CN-CRA-07-55, Rev. 0, "Point Beach GOTHIC Containment Model for LOCA and MSLB Analysis"
8. Reference (DG-M03, Bechtel Piping Class Summary)
9. Seismic Probabilistic Risk Assessment Implementation Guide, EPRI TR 3002000709
10. ASME RA-Sa-2009, Addenda to ASME/ANS RA-S–2008Standard for Level 1/Large Early Release Frequency Probabilistic Risk Assessment for Nuclear Power Plant Applications
11. POD 02131629-02; U1 & U2 SEISMIC
12. Structural Calculations
    a. 11Q0060-C-022, Thermal Evaluation of Unit 1 Containment Dome Truss in Support of Risk Informed LAR
    b. 11Q0060-C-023, Thermal Evaluation of Unit 2 Containment Dome Truss in Support of Risk Informed LAR
    c. 11Q0060-C-024, Seismic Evaluation of Unit 1 Containment Dome Truss in Support of Risk Informed LAR
    d. 11Q0060-C-025, Seismic Evaluation of Unit 2 Containment Dome Truss in Support of Risk Informed LAR
    e. 11Q0060-C-030, Probability of Failure vs. Temperature for Unit 1 Containment Dome Truss in Support of Risk Informed LAR
    f. 11Q0060-C-031, Probability of Failure vs. Temperature for Unit 2 Containment Dome Truss in Support of Risk Informed LAR
13. Modifications EC284214 (Unit 2) and EC285145 (Unit 1)
14. Point Beach EPU: Units 1 (MP) and 2 (WIS) SLB Inside Containment Response. CN-CRA-08-43 Rev 01
15. Point Beach Nuclear Plant Engineering Evaluation 2017-0008, Technical Summary of Methodology and Criteria to Determine the Strength Capacity of the Containment Dome Trusses and Attached/Adjacent Components in Support of License Amendment Request 278
16. Point Beach IPEEE
17. POD 01962836-01; AR: 01962836, Unit 1 Dome Truss and Containment Gap is Less than Expected (Unit 2 Extent of Condition)
18. POD 01986553-01; AR: 01986553 Unit 2 Dome Truss to Containment Liner Gap As-Found Measurements
19. AR # 02193313, LEVEL 1 ASSESSMENT - Background Document for Point Beach All Hazards Table Evaluation.
20. PBN-BFJR-14-013, Point Beach Seismic CDF Estimate

ATTACHMENT A

CONTAINMENT TEMPERATURE END STATE FAULT TREES


270 DEGREES F ALL MITIGATING TRAINS SUCCESSFUL FW ISO SUCCESSFUL
270_ALL_MITIGATING_SUCCESSFUL
9.87E-01
0 OF 2 ACTUATION TRAINS FAIL
0_OF_2_ACT_FAIL
9.99E-01
0 OF 2 CONTAINMENT SPRAY TRAINS FAIL
0_OF_2_CS_FAIL
9.90E-01
0 OF 4 CONTAINMENT FAN COOLERS FAIL
0_OF_4_CFC_FAIL
9.99E-01
0 OF 2 FEEDWATER ISOLATION VALVES FAIL
0_OF_2_FIV_FAIL
9.99E-01
Case 1
270 DEGREES 1 OF 2 TRAINS OF MITIGATION SUCCESSFUL. FW ISO SUCCESSSFUL
270
3.10E-08
1 OF 2 ACTUATION TRAINS FAIL
1_OF_2_ACT_FAIL
1.00E-03
1 OF 2 CONTAINMENT SPRAY TRAINS FAIL
1_OF_2_CS_FAIL
1.00E-02
2 OF 4 CONTAINMENT FAN COOLERS FAIL
2_OF_4_CFC_FAIL
1.00E-03
0 OF 2 FEEDWATER ISOLATION VALVES FAIL
0_OF_2_FIV_FAIL
9.99E-01
Case 2
277.8 DEGREES F ALL MITIGATING TRAINS SUCCESSFUL FW ISO SUCCESSFUL
277_ALL_MITIGATING_SUCCESSFUL
1.09E-03
0 OF 2 ACTUATION TRAINS FAIL
0_OF_2_ACT_FAIL
9.99E-01
0 OF 2 CONTAINMENT SPRAY TRAINS FAIL
0_OF_2_CS_FAIL
9.90E-01
0 OF 4 CONTAINMENT FAN COOLERS FAIL
0_OF_4_CFC_FAIL
9.99E-01
1 OF 2 FEEDWATER ISOLATION VALVES FAIL
1_OF_2_FIV_FAIL
1.10E-03
Case 3
>280 DEGREES 1 OF 2 TRAINS SUCCESSFUL, CS FAILS. FW ISO SUCCESSSFUL
>280_NO_CS
4.00E-09
1 OF 2 ACTUATION TRAINS FAIL
1_OF_2_ACT_FAIL
1.00E-03
2 OF 2 CONTAINMENT SPRAY TRAINS FAIL
2_OF_2_CS_FAIL
1.00E-03
2 OF 4 CONTAINMENT FAN COOLERS FAIL
2_OF_4_CFC_FAIL
1.00E-03
0 OF 2 FEEDWATER ISOLATION VALVES FAIL
0_OF_2_FIV_FAIL
9.99E-01
Case 4
>280 DEGREES 1 OF 2 TRAINS SUCCESSFUL, CFC FAILS. FW ISO SUCCESSSFUL
>280_NO_CFC
1.21E-08
1 OF 2 ACTUATION TRAINS FAIL
1_OF_2_ACT_FAIL
1.00E-03
1 OF 2 CONTAINMENT SPRAY TRAINS FAIL
1_OF_2_CS_FAIL
1.00E-02
3 OF 4 CONTAINMENT FAN COOLERS FAIL
3_OF_4_CFC_FAIL
1.00E-04
0 OF 2 FEEDWATER ISOLATION VALVES FAIL
0_OF_2_FIV_FAIL
9.99E-01
Case 5

The following failure probabilities were developed to be considered in the analysis:

Failure Probabilities of Mitigating Systems		
Failure	**Mean**	**Basis**
FIV	1.1E-3	*NUREG-CR/6928, Table 5-1, AOV FTO/C*
1 of 2 ACT Train	< 1E-3	*PRA*
1 of 4 CFC	~1E-2	*CS and CFC are not included in our RG 1.200 model since they are not needed for LERF at Point Beach. System fault trees were built for CFC and CS to estimate the mean failure probabilities.*
1 of 2 CS	~1E-2	
2 of 4 CFC	~1E-3	
2 of 2 CS	~1E-3	
1 CFC and 1 CS	<1E-3	
2 CFC and 1 CS	<1E-4	
1 CFC and 2 CS	<1E-4	
Loss of AFW	Low	*Gate GAFW300, GAFW110 needs to flow to one SG given SBIC and with & without operator actions (auto)*
MSIVs	<1E-5	*Gate GMS1100, GMS1281 (auto only)*
IR	2.7E-2	*Gate G-ISGTR-1, IRB-INDUCED-SGTR*
Additional combinations	1E-5	*Judgment*

ATTACHMENT B HUMAN RELIABILITY ANALYSIS (HRA)

Based on the event tree evaluation, the operator action to initiate F&B cooling from the control room is the most critical action for mitigation of the consequences of a failed CT. The main steam vent piping (1.5-inch line) is vulnerable to CT failure; therefore, it is likely a small steam break will occur on both steam generators. This is assumed to result in isolation of the steam generators and the need for F&B cooling.

The baseline HEP for the internal events PRA is 5E-2. The internal events HEP was conservatively not modified for the CT to take credit for the NFPA 805 nitrogen supply modification to the PORVs inside containment [Ref 11]. Since the 24-hour nitrogen supply will be inside containment, the operators will no longer be required to reset containment isolation to restore motive force to the PORVs. The internal events HEP was conservatively not modified to take credit for the possible loss of offsite power causing the reactor coolant pumps to trip[17].

The HEP multiplier approach is applied to account for seismic HRA Performance Shaping Factors (PSFs): time available, seismic intensity and location of operator action. Usually, a table is developed to address all these PSF, but in this case the table can be simplified, because the operator action is in the control room and at least 30 minutes to take this action. This leaves seismic intensity to be addressed, given > 30 minutes after the seismic event and actions only in the control room:

SIMPLIFIED PSF ADJUSTMENTS		
Seismic Range	**HEP**	**Basis**
< 0.23g	**5E-2**	based on internal event PRA
0.23g to 0.56g	**0.15**	Factor of 3 multiplier
0.56g to 1g	**0.5**	Factor of 10 multiplier
> 1g	**1.0**	Guaranteed Failure

The HEP multiplier approach provides a systematic way to evaluate human failure events against selected key factors affecting operator performance.

To identify the HEP multipliers the following key factors are considered:

- **Intensity of Seismic Event (based on PGA):** The higher the intensity of the seismic event, the more likely there will be distracting conditions (e.g., steam leaks, sound effects from the event, confusion, concerns for other personnel etc.) and increased stress. It is assumed there are no hazards, masonry walls or weak structural elements at PB in the vicinity of control room.
- **Location of the operator action and Access to Equipment Location**: The location where the required action must be taken (in control room versus ex-control room) can become a key impact on human reliability after a seismic event. For example, after a very large earthquake where local damage has occurred (e.g., block walls have failed) and an action is required in this structure; access and successful operation may be difficult due to debris and other obstacles. Conversely, operator actions in the control room may not be affected. F&B alignment takes place in the most optimum location, the control room, and does not require local actions.
- **Time after the seismic event**: The longer the time between occurrence of the seismic event and the need for operator action, the more likely it is that the operator will recover from the initial stresses of the event. Performance shaping factors (PSF), as currently utilized in PRAs, are most significant in the first few minutes. It

[17] *If the reset of containment isolation is removed, the HEP drops to 3.64E-2, not a significant change.*

is assumed that this initial stress would be reduced as time moves forward, situations are analyzed, operational control of equipment is re-established, tank sloshing subsides and additional resources become available. F&B would be required more than 30 minutes after the seismic event, which is not the most optimal (e.g., several hours), but still sufficient time to recover from the event.

Review of the IPEEE did not identify vulnerabilities with the control room, cabinets and instruments required to support control room operation. Although the IPEEE identified cable trays in the cable spreading room as a vulnerability, the cable trays have been modified to be more seismically robust[Ref 16]

Other HEPs such as sump recirculation occur much later in time, greater than 5 hours, and after an intervening success. The only operator actions for sump recirculation outside the control room are manually closing the SI-897A or SI-897B valves and component cooling water realignment. These actions take place on the El 8, bottom floor, of the PAB, a seismic structure and are not likely to encounter significant obstacles following an earthquake. Given the extensive time available and the location of the operator actions, no adjustment to the recirculation HEP is necessary.

A sensitivity analysis was performed on the HEP to determine the effect of crediting the planned PORV nitrogen supply modifications [Ref 13]. The PORV nitrogen supply is located inside containment and therefore the step in the procedure that requires realignment of instrument air to the containment for F&B cooling can be removed. Removal of the restoration of instrument air to containment (Step 36 in the current procedure) reduced the HEP value from **0.05** to **0.04**.

As a sensitivity the factors from the simplified PSF table and reduced HEP of 0.04 were applied to the two demonstrably conservative seismic cases with the following results:

SEISMIC CASE	ΔCDF with HEP PSF sensitivity	ΔCDF with HEP=0.05	ΔCDF with HEP=0.04
Demonstrably Conservative	3.97E-07	1.34E-07	1.13E-07
Demonstrably Conservative convolved with IPEEE	2.26E-07	9.38E-08	7.88E-08

TOTAL ΔCDF SEISMIC +THERMAL			
TOTAL CASE	ΔCDF with HEP PSF sensitivity	ΔCDF with HEP=0.05	ΔCDF with HEP=0.04
Demonstrably Conservative	4.37E-07	1.74E-07	1.53E-07
Demonstrably Conservative convolved with IPEEE	2.66E-07	1.34E-07	1.19E-07

Application of the conservative PSF factors has a factor of 2.5 impact on the demonstrably conservative case ΔCDF and factor of 2 impact on the convolved case. This increase does will not change the applicable RG 1.174 Region and as such the conclusions of this evaluation. Applying an HEP of 0.04 reduces the ΔCDF by ~11%.

ATTACHMENT C ASME/RG 1.200 PRA QUALITY SUPPORTING REQUIREMENTS

REQUIREMENTS FOR OTHER EXTERNAL HAZARDS: REQUIREMENTS FOR SCREENING AND CONSERVATIVE ANALYSIS (EXT)	
HIGH LEVEL SUPPORTING REQUIREMENTS	**STATEMENT OF COMPLIANCE**
HLR-EXT-A All potential external hazards (i.e., all natural and man-made hazards) that may affect the site shall be identified.	Only seismic and thermal hazards apply to this evaluation.
HLR-EXT-B Preliminary screening, if used, shall be performed using a defined set of screening criteria.	Screening criteria is define in section 1.0
HLR-EXT-C A bounding or demonstrably conservative analysis, if used for screening, shall be performed using defined quantitative screening criteria.	Both bounding and demonstrably conservative analysis were perform using quantitative screening criteria as described in section 1.0
HLR-EXT-D The basis for the screening out of an external hazard shall be confirmed through a walkdown of the plant and its surroundings.	Walkdowns were only used to confirm modifications credited in this evaluation were located and protected in a way that avoid damage from debris generated by a failed CT.
HLR-EXT-E Documentation of the screening out of an external hazard shall be consistent with the applicable supporting requirements.	This evaluation meets the applicable documentation requirements.

REQUIREMENTS FOR OTHER EXTERNAL HAZARDS: REQUIREMENTS FOR SCREENING AND CONSERVATIVE ANALYSIS (EXT)	
SUPPORTING REQUIREMENTS FOR HLR-EXT-A	**STATEMENT OF COMPLIANCE**
EXT-A1 In the list of external hazards, INCLUDE as a minimum those that are enumerated in the PRA Procedures Guide, NUREG/CR-2300 [6-1] and NUREG-1407 [6-3] and examined in past studies such as the NUREG-1150 analyses [6-4]. Nonmandatory Appendix 6-A contains the list adapted from NUREG/CR-2300, and this list provides one acceptable way to meet this requirement.	*Does not apply. This evaluation focuses on a seismic evaluation of the Point Beach Unit 1 and Unit 2 Construction Trusses.*
EXT-A2 SUPPLEMENT the list considered in (EXT-A1) with any site-specific and plant-unique external hazards.	*This evaluation is a site-specific and plant unique seismic evaluation of the Point Beach Unit 1 and Unit 2 Construction Trusses.*

REQUIREMENTS FOR OTHER EXTERNAL HAZARDS: REQUIREMENTS FOR SCREENING AND CONSERVATIVE ANALYSIS (EXT)	
SUPPORTING REQUIREMENTS FOR HLR-EXT-B	**STATEMENT OF COMPLIANCE**
EXT-B1 Initial Preliminary Screening: For screening out an external hazard, any one of the following five screening criteria provides as an acceptable basis: *Criterion 1:* The event is of equal or lesser damage potential than the events for which the plant has been designed. This requires an evaluation of plant design bases in order to estimate the resistance of plant structures and systems to a particular external hazard. *Criterion 2:* The event has a significantly lower mean frequency of occurrence than another event, taking into account the uncertainties in the estimates of both frequencies, and the event could not result in worse consequences than the consequences from the other event. *Criterion 3:* The event cannot occur close enough to the plant to affect it. This criterion must be applied taking into account the range of magnitudes of the event for the recurrence frequencies of interest. *Criterion 4:* The event is included in the definition of another event. *Criterion 5:* The event is slow in developing, and it can be demonstrated that there is sufficient time to eliminate the source of the threat or to provide an adequate response.	*None of these screening criteria apply.*
EXT-B2 Second Preliminary Screening: For screening out an external hazard other than seismic events, the following screening criterion provides an acceptable basis. The criterion is that the design basis for the event meets the criteria in the U.S. Nuclear Regulatory Commission 1975 Standard Review Plan [6-2].	*This screening criterion does not apply.*
EXT-B3 BASE the application of the screening criteria for a given external hazard on a review of information on the plant's design hazard and licensing basis relevant to that event.	*Relevant license basis requirements were reviewed and incorporated/considered into the structural calculations were relevant .*
EXT-B4 REVIEW any significant changes since the U.S. Nuclear Regulatory Commission operating license was issued. In particular, review all of the following: *(a)* military and industrial facilities within 8 km of the site *(b)* on-site storage or other activities involving hazardous materials *(c)* nearby transportation *(d)* any other developments that could affect the original design conditions	*GMRS was based on latest site-specific hazard as documented in ML14090A275 (which was obtained from the work performed by Lettis Consultants) and accepted by the NRC in ML15211A593.*

REQUIREMENTS FOR OTHER EXTERNAL HAZARDS: REQUIREMENTS FOR SCREENING AND CONSERVATIVE ANALYSIS (EXT)	
SUPPORTING REQUIREMENTS FOR HLR-EXT-C	**STATEMENT OF COMPLIANCE**
EXT-C1 For screening out an external hazard, any one of the following three screening criteria provides an acceptable basis for bounding analysis or demonstrably conservative analysis. *Criterion A:* The current design-basis-hazard event cannot cause a core damage accident. *Criterion B:* The current design-basis-hazard event has a mean frequency <10-5/yr, and the	***Criterion B** was applied to the demonstrably conservative analysis which has a mean frequency of < 10-5/yr and a mean CCDP of <10-1.* ***Criterion C** also applies to the demonstrably conservative analysis which has a core damage frequency of 3.93E-07 for seismic +4.45E-08 for thermal which equals **4.39E-07 total CDF.***

mean value of the conditional core damage probability (CCDP) is assessed to be <10-1. *Criterion C:* The core damage frequency, calculated using a bounding or demonstrably conservative analysis, has a mean frequency <10-6/yr.	
EXT-C2 BASE the estimation of the mean frequency and the other parameters of the design-basis hazard or the bound on them using hazard modeling and recent data (e.g., annual maximum wind speeds at the site, aircraft activity in the vicinity, or precipitation data).	*GMRS was based on latest site-specific hazard as documented in ML14090A275 (which was obtained from the work performed by Lettis Consultants) and accepted by the NRC in ML15211A593.*
EXT-C3 In estimating the mean conditional core damage probability (CCDP), USE a bounding analysis or a demonstrably conservative analysis that employs a systems model of the plant that meets the systems-analysis requirements in Part 2 insofar as they apply [6-6].	*A bounding analysis and demonstrably conservative analysis were used. Since mitigating systems and operator actions are not credited, neither a full-scope nor partial PRA model is needed to perform this assessment. Only seismic and thermal hazard and fragility data are used to calculate core damage probability. Section 2.0 fully describes the scope of the bounding analysis.* *The demonstrably conservative analysis used simplified event trees that show the sequences considered and success and failure probabilities of critical systems and components. These event trees were developed assuming a seismic or thermal event caused CT failure. Non-seismic SSCs like air compressors and instrument air lines are considered failed and not included. Seismic SSCs outside containment are assumed not to fail since they are designed for the DBE and will not fail due to thermal transients inside containment. Section 5.0 fully describes the scope of the demonstrably conservative analysis.*
EXT-C4 IDENTIFY those SSCs required to maintain the plant in operation or that are required to respond to an initiating event to prevent core damage, that are vulnerable to the hazard, and determine their failure modes.	*The event trees developed in section 5 identify SSCs required to prevent core damage in the event the CT fails.*
EXT-C5 ESTIMATE the CCDP taking into account the initiating events caused by the hazard, and the systems or functions rendered unavailable. Modifying the internal-events PRA model as appropriate, using conservative assessments of the impact of the hazard (fragility analysis), is an acceptable approach.	*The event trees in section 5 were used to calculate CCDP for seismic and relevant thermal hazards. Conservative values were used for the failure probabilities for the SSCs credited.*
EXT-C6 BASE the estimation of the mean core damage frequency developed here on models and data that are either realistic or demonstrably conservative. This includes not only the hazard analysis but also any fragility analysis that is applicable	*The CDF developed is based on modeling that is demonstrably conservative as described in section 5.0.*
EXT-C7 If none of the screening criteria in this entire Part 6 can be met for a given external hazard, then PERFORM additional analysis. (See Parts 7, 8, and 9.)	*The screening criteria for the seismic hazard is met. However the ΔCDF and ΔLERF were also calculated to show compliance to RG 1.174.*

REQUIREMENTS FOR OTHER EXTERNAL HAZARDS: REQUIREMENTS FOR SCREENING AND CONSERVATIVE ANALYSIS (EXT)	
SUPPORTING REQUIREMENTS FOR HLR-EXT-D	**STATEMENT OF COMPLIANCE**
EXT-D1 CONFIRM the basis for the screening out of an external hazard through a walkdown of the plant and its surroundings	*Although the seismic hazard met the screening criteria the evaluation also showed compliance to RG 1.174 criteria. Confirmatory walkdowns were performed to assure that Modifications EC284214 (Unit 2) and EC285145 (Unit 1) [Ref 13] that install seismic category 1 nitrogen tanks and piping to supply the PORVs in the event of a loss of instrument air to the PORVs are adequately protected from debris that result from a failed CT.*
EXT-D2 If the screening out of any specific external hazard depends on the specific plant layout, then CONFIRM that layout with a walkdown. For most external hazards, this typically means a walkdown that evaluates the site layout outside the plant buildings as well as inside.	*Not applicable – hazard not screened out.*

REQUIREMENTS FOR OTHER EXTERNAL HAZARDS: REQUIREMENTS FOR SCREENING AND CONSERVATIVE ANALYSIS (EXT)	
SUPPORTING REQUIREMENTS FOR HLR-EXT-E	**STATEMENT OF COMPLIANCE**
EXT-E1 DOCUMENT the external hazard screening and conservative analyses in a manner that facilitates PRA applications, upgrades, and peer review.	*This evaluation complies with this requirement.*
EXT-E2 DOCUMENT the process used in the external hazard screening and conservative analyses. For example, this documentation typically includes a description of: *(a)* the approach used for the screening (preliminary screening or demonstrably conservative analysis) and the screening criteria used for each external hazard that is screened out, *(b)* any engineering or other analysis performed to support the screening out of an external hazard or in the conservative assessment of an external hazard.	*This document fully describes the risk analysis and fully references all the structural inputs utilized.*

HIGH LEVEL SUPPORTING REQUIREMENTS - HAZARDS	
HIGH LEVEL SUPPORTING REQUIREMENTS	**STATEMENT OF COMPLIANCE**
HLR-SHA-A The frequency of earthquakes at the site shall be based on a site-specific probabilistic seismic hazard analysis (existing or new) that reflects the composite distribution of the informed technical community. The level of analysis shall be determined based on theintended application and on site-specific complexity.	GMRS based on site-specific hazard as documented in ML14090A275 (which was obtained from the work performed by Lettis Consultants) and accepted by the NRC in ML15211A593.
HLR-SHA-B To provide inputs to the probabilistic seismic hazard analysis, a comprehensive up-to-date database, including geological, seismological, and geophysical data; local site topography; and surficial geologic and geotechnical site properties, shall be compiled. A catalog of historical, instrumental, and paleoseismicity information shall also be compiled.	

HLR-SHA-C To account for the frequency of occurrence of earthquakes in the site region, the probabilistic seismic hazard analysis shall examine all credible sources of potentially damaging earthquakes. Both the aleatory and epistemic uncertainties shall be addressed in characterizing the seismic sources.	The Lettis Consultants developed report, submitted as part of correspondence NRC 2014-0024, dated 03/31/2014, *NextEra Energy Point Beach, LLC Seismic Hazard and Screening Report (CEUS Sites), Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*, (ML14090A275) should be consulted for further detail. For information beyond the scope of the report, Lettis Consultants should be consulted.
HLR-SHA-D The probabilistic seismic hazard analysis shall examine credible mechanisms influencing estimates of vibratory ground motion that can occur at a site given the occurrence of an earthquake of a certain magnitude at a certain location. Both the aleatory and epistemic uncertainties shall be addressed in characterizing the ground motion propagation.	
HLR-SHA-E The probabilistic seismic hazard analysis shall account for the effects of local site response.	
HLR-SHA-F Uncertainties in each step of the hazard analysis shall be propagated and displayed in the final quantification of hazard estimates for the site. The results shall include fractile hazard curves, median and mean hazard curves, and uniform hazard response spectra. For certain applications, the probabilistic seismic hazard analysis shall include seismic source deaggregation and magnitude-distance deaggregation.	
HLR-SHA-G For further use in the seismic PRA, the spectral shape shall be based on a site-specific evaluation taking into account the contributions of deaggregated magnitude-distance results of the probabilistic seismic hazard analysis. Broad-band, smooth spectral shapes, such as those presented in NUREG/CR-0098 (for lower-seismicity sites such as most of those east of the U.S. Rocky Mountains) are also acceptable if they are shown to be appropriate for the site. The use of uniform hazard response spectra is also acceptable unless evidence comes to light that would challenge these uniform hazard spectral shapes.	
HLR-SHA-H When use is made of an existing study for probabilistic seismic hazard analysis purposes, it shall be confirmed that the basic data and interpretations are still valid in light of current information, the study meets the requirements outlined in A through G above, and the study is suitable for the intended application.	
HLR-SHA-I A screening analysis shall be performed to assess whether in addition to the vibratory ground motion, other seismic hazards, such as fault displacement, landslide, soil liquefaction, or soil settlement, need to be included in the seismic PRA for the specific application. If so, the seismic PRA shall address the effect of these hazards through assessment of the frequency of hazard occurrence or the magnitude of hazard consequences, or both.	
HLR-SHA-J Documentation of the probabilistic seismic hazard analysis shall be consistent with the applicable supporting requirements.	

SEISMIC SUPPORTING REQUIREMENTS -HAZARDS	
HIGH LEVEL SUPPORTING REQUIREMENTS	**STATEMENT OF COMPLIANCE**
SHA-A1 In performing the probabilistic seismic hazard analysis (PSHA), BASE it on, and MAKE it consist of, the collection and evaluation of available information and data, consideration of the uncertainties in each element of the PSHA, and a defined process and documentation to make the PSHA traceable. *The guidance and process given in* NUREG/CR-6372 *address the above requirement and MAY be used as an acceptable methodology. In general, Levels 1 and 2 of these references correspond to Capability Category I, Levels 2 and 3 to Capability Category II, and Levels 3 and 4 to Capability Category III. The distinction between the consideration of uncertainties (for Capability Category I) and the evaluation of them (Capability Categories II and III) is important. The latter means a numerical evaluation.*	The PRA methods used in this evaluation a bounding analysis and demonstrably conservative analysis. The bounding analysis used the fragility data from the structural calculations and applied a CCDP of 1.0 to the resulting failure frequency. The demonstrably conservative analysis used the same fragility data but applied failure probability to SSCs that were relevant to the sequences resulting from a failed CT.
SHA-A2 As the parameter to characterize both hazard and fragilities, USE the spectral accelerations, or the average spectral acceleration over a selected band of frequencies, or peak ground acceleration. *While the use of peak ground acceleration as a parameter to characterize both hazard and fragility has been a common practice in the past and is acceptable, the use of spectral accelerations is preferable.*	The fragility is determined in reference to the PGA as documented in calculation 11Q0060-C-028.
SHA-A3 In the selection of frequencies to determine spectral accelerations or average spectral acceleration, CAPTURE the frequencies of those structures, systems, or components, or a combination thereof that are significant in the PRA results and insights.	The seismic demand is based on the frequency of the structure as shown in calculations 11Q0060-C-024, -025, -032, & -033.
SHA-A4 In developing the probabilistic seismic hazard analysis results, whether they are characterized by spectral accelerations, peak ground accelerations, or both, EXTEND them to large-enough values (consistent with the physical data and interpretations) so that the truncation does not produce unstable final numerical results, such as core damage frequency, and the delineation and ranking of seismic-initiated sequences are not affected. *It is necessary to make sure that the hazard estimation is carried out to large-enough values (consistent with the physical data and interpretations) so that when convolved with the plant or component level fragility, the resulting failure frequencies are robust estimates and do not change if the acceleration range is extended. A sensitivity study can be conducted to define the upper-bound value. NUREG-1407 provides the additional guidance. Peer review needs to be attentive to this aspect.*	The PRA evaluation used "demonstrably conservative and bounding methods which did not require the use of PRA quantification tools that set truncation limits.
SHA-A5 SPECIFY a lower-bound magnitude (or probabilistically defined characterization of magnitudes based on a damage	Calculation 11Q0060-C-037 evaluate the

parameter) for use in the hazard analysis, such that earthquakes of magnitude less than this value are not expected to cause significant damage to the engineered structures or equipment. *The value of the lower-bound magnitude used in analyzing the site-specific hazard is based on engineering considerations [5-26][18]. Based on the evaluation of earthquake experience data, earthquakes with magnitudes less than 5.0 are not expected to cause damage to safety-related structures, or systems, or components, or a combination thereof. A lower-bound magnitude value of 5.0 was used for both the Lawrence Livermore National Laboratory and Electric Power Research Institute studies. The latest research in this area recommends using a probabilistically defined characterization of what magnitudes are expected to cause damage based on the Cumulative Absolute Velocity (CAV) parameter. Note that this lower bound applies only to the magnitude range considered in the final hazard quantification, not to the characterization and determination of seismicity parameters for the sources. The choice of magnitude scale should be consistent with the one used in the ground motion attenuation models and should be documented.*	Containment Dome Truss structures (including justification for attached and adjacent SSCs) for a bounding seismic acceleration, below which the truss structures, attached components, and adjacent structures will maintain stresses within an elastic limit (i.e. not resulting in significant damage to the truss structures).
SHA-B1 In performing the probabilistic seismic hazard analysis (PSHA), BASE it on available or developed geological, seismological, geophysical, and geotechnical databases that reflect the current state of the knowledge and that are used by experts/analysts to develop interpretations and inputs to the PSHA. *It is important that a comprehensive database be shared and used by all experts in developing the interpretations. The availability of the database also facilitates the review process. RG 1.165 gives acceptable guidance on the scope and types of data required for use in the seismic source characterization, ground motion modeling, and local site response evaluations to meet this requirement.*	The Lettis Consultants developed report, submitted as part of correspondence NRC 2014-0024, dated 03/31/2014, *NextEra Energy Point Beach, LLC Seismic Hazard and Screening Report (CEUS Sites), Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*, (ML14090A275) should be consulted for further detail. For information beyond the scope of the report, Lettis Consultants should be consulted.
SHA-B2 ENSURE that the database and information used are adequate to characterize all credible seismic sources that may contribute to the frequency of occurrence of vibratory ground motion at the site, considering regional attenuation of ground motions and local site effects. If the existing probabilistic seismic hazard analysis (PSHA) studies are to be used in the seismic PRA, ENSURE that any new data or interpretations that could affect the PSHA are adequately incorporated in the existing databases and analysis. *RG 1.165 defines four levels of investigations, with the degree of their detail based on distance from the site, the nature of the Quaternary tectonic regime, the geological complexity of the site and region, the existence of potential seismic sources, the nature of sources, the potential for surface deformation, etc. This guidance can be used to determine scope and size of region for investigations. The guidance in NUREG-0800, Section 2-5.2, may be used to meet this requirement.*	
SHA-B3 As a part of the database used, INCLUDE a catalog of historically reported, geologically identified, and instrumentally recorded earthquakes. USE reference NUREG-0800, Section 2-5.2, requirements or equivalent. *In general, the catalog typically includes events of size modified Mercalli intensity (MMI) or equivalent greater than or equal to IV and magnitude greater than or equal to 3.0 that have occurred within a radius of 320 km of a site, reference* NUREG-0800, Section 2-5.2. *For the earthquakes listed, the catalog typically contains information such as event date and time, epicentral location, earthquake magnitudes (measured and calculated), magnitude uncertainty, uncertainty in the event location, epicentral intensity, intensity uncertainty, hypocentral depth, references, and data sources.*	
SHA-C1 In the probabilistic seismic hazard analysis, EXAMINE all potential sources of earthquakes that affect the probabilistic hazard at the site. BASE the identification and characterization of seismic sources on regional and site geological and geophysical data, historical and instrumental seismicity data, the regional stress field, and geological evidence of prehistoric earthquakes.	
SHA-C2 ENSURE that any expert elicitation process used to characterize the seismic sources is compatible with the level of analysis discussed in Requirement HA-A, and FOLLOW a structured approach.	
SHA-C3 The seismic sources are characterized by source location and geometry, maximum earthquake magnitude, and earthquake recurrence. INCLUDE the aleatory and epistemic uncertain ties explicitly in these characterizations.	
SHA-C4 If an existing probabilistic seismic hazard analysis study is used, SHOW that any seismic sources that were previously unknown or uncharacterized are not significant, or INCLUDE them in a revision of the hazard estimates.	
SHA-D1 ACCOUNT in the probabilistic seismic hazard analysis for *(a)* credible mechanisms governing estimates of vibratory ground motion that can occur at a site *(b)* regional and site-specific geological, geophysical, and geotechnical data and historical and instrumental seismicity data (including strong motion data) *(c)* current attenuation models in the ground motion estimates	
SHA-D2 ENSURE that any expert elicitation process used to characterize the ground motion is compatible with the level of analysis discussed in Requirement SHA-A, and FOLLOW a structured approach.	
SHA-D3	

18 *"Final Report of the Diablo Canyon Long Term Seismic Program," Pacific Gas and Electric Company; available from theU.S. Nuclear Regulatory Commission, Dockets 50-275 and 50-323 (1988)*

ADDRESS both the aleatory and epistemic uncertainties in the ground motion characterization in accordance with the level of analysis identified for Requirement SHA-A.	
SHA-D4 If an existing probabilistic seismic hazard analysis study is used, SHOW that any ground motion models or new information that were previously unused or unknown are not significant, or INCLUDE them in a revision of the hazard estimates.	
SHA-E1 ACCOUNT in the probabilistic seismic hazard analysis for the effects of site topography, surficial geologic deposits, and site geotechnical properties on ground motions at the site.	
SHA-E2 ADDRESS both the aleatory and epistemic uncertainties in the local site response analysis.	
SHA-F1 In the final quantification of the seismic hazard, INCLUDE and DISPLAY the propagation of both aleatory and epistemic uncertainties.	
SHA-F2 In the probabilistic seismic hazard analysis, INCLUDE appropriate sensitivity studies and intermediate results to identify factors that are important to the site hazard and that make the analysis traceable.	
SHA-F3 DEVELOP the following results as a part of the quantification process, compatible with needs for the level of analysis determined in (HLR-SHA-A): *(a)* fractile and mean hazard curves for each ground motion parameter considered in the probabilistic seismic hazard analysis *(b)* fractile and mean uniform hazard response spectrum	
SHA-G1 BASE the response spectral shape used in the seismic PRA on site-specific evaluations performed for the probabilistic seismic hazard analysis. REFLECT or BOUND the site-specific considerations.	
SHA-H Use of existing studies allowed. *When using the Lawrence Livermore National Laboratory/U.S. Nuclear Regulatory Commission [5-24] or Electric Power Research Institute [5-25] hazard studies, or another study done to a comparable technical level, the intent of this requirement is not to repeat the entire hazard exercise or calculations, unless new information and interpretations that affect the site have been established and affect the usefulness of the seismic PRA for the intended application. Depending upon the application, sensitivity studies, modest extensions of the existing analysis, or approximate estimates of the differences between using an existing hazard study and applying the newer one may be sufficient. Additionally, an educated assessment may be sufficient to demonstrate that the impact on the application of information or data that is less extensive than a new hazard study is not significant.*	

HIGH LEVEL REQUIREMENTS - FRAGILITY	
HIGH LEVEL SUPPORTING REQUIREMENTS	Screening of high-seismic-capacity components was not required.
HLR-SFR-A The seismic-fragility evaluation shall be performed to estimate plant-specific, realistic seismic fragilities of structures, or systems, or components, or combination thereof whose failure may contribute to core damage or large early release, or both.	The seismic demand is based on a realistic seismic response captured through a soil-structure interaction analysis as documented in calculation 11Q0060-C-027.
HLR-SFR-B If screening of high-seismic-capacity components is performed, the basis for the screening shall be fully described.	The seismic fragilities of the CDTs are based on the critical failure mode based on the highest loaded component as documented in calculations 11Q0060-C-024 & -025.
HLR-SFR-C The seismic-fragility evaluation shall be based on realistic seismic response that the SSCs experience at their failure levels.	The analysis of the CDTs account for information obtained from walkdowns as documented in calculations 11Q0060-C-024 & -025.
HLR-SFR-D The seismic-fragility evaluation shall be performed for critical failure modes of structures, systems, or components, or a combination thereof such as structural failure modes and functional failure modes identified through the review of plant design documents, supplemented as needed by earthquake experience data, fragility test data, generic qualification test data, and a walkdown.	The seismic fragilities are calculated based on generic parameters that are considered to be bounding as documented in calculation 11Q0060-C-028.
HLR-SFR-E The seismic-fragility evaluation shall incorporate the findings of a detailed walkdown of the plant focusing on the anchorage, lateral seismic support, and potential systems interactions.	The seismic fragility is documented in calculation 11Q0060-C-028.
HLR-SFR-F The calculation of seismic-fragility parameters such as median capacity and variabilities shall be based on plant-specific data supplemented as needed by earthquake experience data, fragility test data, and generic qualification test data. Use of such generic data shall be justified.	The realistic seismic fragility of the CDTs is calculated as documented in calculations 11Q0060-C-024, -025, -028, -032, & -033.
HLR-SFR-G Documentation of the seismic-fragility evaluation shall be consistent with the applicable supporting requirements.	Screening of high-seismic-capacity components was not required.

SEISMIC SUPPORTING REQUIREMENTS - FRAGILITY	STATEMENT OF COMPLIANCE
SFR-A1 CALCULATE seismic fragilities for SSCs identified by the systems analysis (see Requirement SPR-D1). *NOTE: (1) Seismic fragilities are needed for SSCs identified by the systems analysis that are modeled in the event trees and fault trees. Failure of one or more of these may contribute to core damage or large early release, or both. Requirements for developing this list of SSCs are given under the Systems Analysis section (see Requirement SPR-D1). See also the Requirement HLR-SFR-B on screening.*	Seismic fragilities are documented in calculation 11Q0060-C-028 and are calculated for the CDTs and the liner/wall. The seismic fragilities for attached components are shown to be bounded.
SFR-A2 CALCULATE the seismic fragilities based on plant-specific data, and ENSURE that they are realistic (median with uncertainties). *NOTE: (2) The objective of a seismic PRA is to obtain a realistic seismic risk profile for the plant using plant-specific and site-specific data. It has been demonstrated in several seismic PRAs that the risk estimates and insights on seismic vulnerabilities are very plant specific, even varying between supposedly identical units at a multiunit plant. To minimize the effort on nonsignificant items and to focus the resources on the more critical aspects of the seismic PRA, certain high-seismic-capacity components are screened out using generic data (e.g., fragility test data, generic seismic qualification test data, and earthquake experience data).*	Site specific data was used to determine the GMRS as documented in ML14090A275 (which was obtained from the work performed by Lettis Consultants) and accepted by the NRC in ML15211A593. The in-structure response spectra was based on site-specific soil data as documented in calculation 11Q0060-C-027.
SFR-B1 SCREEN OUT high-seismic-capacity components only if the components' failures can be considered as fully independent of the remaining components. *NOTES: (1) When screening of high-seismic-capacity components is performed, the basis for screening and supporting documents is to be fully described. Guidance given in EPRI NP-6041-SL, Rev. 1 [5-3] and NUREG/CR-4334 [5-4] may be used to screen out high-seismic-capacity components after satisfying the caveats. Note that the screening guidance in these documents has been developed generally for U.S.-vendored equipment and based on U.S. seismic design practice. Care should be used in applying the screening criteria for other situations. The use of generic fragility information is acceptable for screening if the SSCs can be shown to fall within the envelope of the generic fragility caveats. The screening level chosen should be based on the seismic hazard at the site and on the plant seismic design basis and should be high enough that the contribution to core damage frequency and large early release frequency from the screened-out components is not significant. (See Requirement SHA-G1.) For a discussion of possible approaches to the selection of the screening level, the reader is referred to reference [5-10]*	Screening out of high-seismic-capacity components was not required.
SFR-C1 ESTIMATE the seismic responses that the components experience at their failure levels on a realistic basis using site-specific earthquake response spectra in three orthogonal directions, anchored to a ground motion parameter such as peak ground acceleration or average spectral acceleration over a given frequency band. *NOTES: (1) NUREG-1407 [5-7] recommends the use of 10,000-yr return period UHS median spectral shapes provided in reference [5-32] along with variability estimates that reflect the site-specific shapes as discussed in Note (1) of Table 5-2.1-8. Any UHS should be used cautiously to ensure that the spectral shape reflects the contributions from dominating events as discussed under Requirement SHA-G1. See Note (1) of Table 5-2.1-8*	The Lettis Consultants developed report, submitted as part of correspondence NRC 2014-0024, dated 03/31/2014, *NextEra Energy Point Beach, LLC Seismic Hazard and Screening Report (CEUS Sites), Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*, (ML14090A275) should be consulted for further detail. For information beyond the scope of the report, Lettis Consultants should be consulted.
SFR-C2 PERFORM probabilistic seismic response analysis taking into account the uncertainties in the input ground motion and site soil properties, and structural parameters, and ESTIMATE joint probability distributions of the responses of different components in the building. *NOTES: (2) For a description of the probabilistic seismic response analysis, the reader is referred to references [5-38] and [5-31].*	The seismic response is calculated using five (5) real motion time histories in each orthogonal direction that are modified to match the GMRS (calculation 11Q0060-C-026). The in-structure seismic response accounts for the variability in soil properties by enveloping the lower estimate, best estimate, and upper estimate of the soil properties, and broadened to account for structural uncertainties (calculation 11Q0060-C-027).
SFR-C3 Addressed in Requirement SFR-C2 *NOTES: (3) The scaling procedures given in reference [5-3] may be used. Scaling of responses from existing analysis is not permitted for Capability Category III.*	N/A
SFR-C4 Addressed in Requirement SFR-C2	N/A
SFR-C5 Addressed in Requirement SFR-C2 *NOTES:(4) Reference [5-10] gives an acceptable method.*	N/A
SFR-C6 Addressed in Requirement SFR-C2 *NOTES: (5) Further details about the basis of this requirement can be found in reference [5-15].*	N/A
SFR-D1 IDENTIFY realistic failure modes of structures (e.g., sliding, overturning, yielding, and excessive drift), equipment	The seismic fragilities of the CDTs are based on the critical failure mode based on the highest loaded

(e.g., anchorage failure, impact with adjacent equipment or structures, bracing failure, and functional failure), and soil (e.g., liquefaction, slope instability, and excessive differential settlement) that interfere with the operability of equipment during or after the earthquake, through a review of the plant design documents and the walkdown. *NOTES: (1) Note that sometimes failure modes such as drift and yielding may be more relevant for the functionality of attached equipment than gross structural failures (e.g., partial collapse or complete collapse)*	component as documented in calculations 11Q0060-C-024 & -025.
SFR-D2 EVALUATE all relevant failure modes identified in Requirement SFR-D1, and EVALUATE fragilities for critical failure modes. *NOTES: (2) Published references and past seismic PRAs could be used as guidance. Examples include references [5-3], [5-10], and [5-26*	The seismic fragilities of the CDTs are based on the critical failure mode based on the highest loaded component as documented in calculations 11Q0060-C-024 & -025.
SFR-E1 CONDUCT a detailed walkdown of the plant, focusing on equipment anchorage, lateral seismic support, spatial interactions, and potential systems interactions (both structural and functional interactions). *NOTES: (1) The seismic walkdown is an important activity in the seismic PRA. The purposes of such a walkdown are to find as-designed, as-built, and as-operated seismic weaknesses in the plant and to ensure that the seismic fragilities are realistic and plant specific. It should be done in sufficient detail and documented in a sufficiently complete fashion so that the subsequent screening or fragility evaluation is traceable. For guidance on walkdowns, the analyst is referred to references [5-3] and [5-4]. (See Requirement SPR-B9.)*	The analysis of the CDTs account for information obtained from walkdowns as documented in calculations 11Q0060-C-024 & -025.
SFR-E2 DOCUMENT the walkdown procedures, walkdown team composition and its members' qualifications, walkdown observations, and conclusions.	Various walkdowns were performed in support of the structural evaluation and documented in the calculations.
SFR-E3 If components are screened out during or following the walkdown, DOCUMENT the basis, including any anchorage calculations that justify such a screening.	Screening of high-seismic-capacity components was not required.
SFR-E4 During the walkdown, EVALUATE the potential for seismically induced fire and flooding by focusing on the issues described in NUREG-1407 [5-7]. *NOTES: (2) Seismically induced fires and floods are to be addressed as described in NUREG-1407 [5-7]. The effects of seismically induced fires and impact of inadvertent actuation of fire protection systems on safety systems should be assessed. The effects of seismically induced external flooding and internal flooding on plant safety should be included. The scope of the evaluation of seismically induced flood, in addition to that of the external sources of water (e.g., tanks and upstream dams), should include the evaluation of some internal flooding that is consistent with the discussion*	Not applicable. This evaluation focuses on the CT only.
SFR-E5 During the walkdown, EVALUATE potential sources of interaction (e.g., II/I issues, impact between cabinets, masonry walls, flammable and combustion sources, flooding, and spray) and consequences of such interactions on equipment contained in the systems model. *NOTES: (3) A "II/I issue" refers to situations where a nonseismically qualified object could fall on and damage a seismically qualified item of safety equipment, and also situations where a low seismic capacity object falls on and damages an SSC item with higher seismic capacity. In such cases, the fragility of the higher capacity SSC may be controlled by the low capacity object.*	Reference 3 evaluated the interaction of CT debris on components below. This applied only to the demonstrably conservative analysis. The bounding analysis assumed that CT failure results in core damage, CCDP = 1.0.
SFR-F1 During the walkdown, EVALUATE potential sources of interaction (e.g., II/I issues, impact between cabinets, masonry walls, flammable and combustion sources, flooding, and spray) and consequences of such interactions on equipment contained in the systems model. *NOTES: (3) A "II/I issue" refers to situations where a nonseismically qualified object could fall on and damage a seismically qualified item of safety equipment, and also situations where a low seismic capacity object falls on and damages an SSC item with higher seismic capacity. In such cases, the fragility of the higher capacity SSC may be controlled by the low capacity object.*	
SFR-F2 For all SSCs that appear in the significant accident sequences, ENSURE that they have site-specific fragility parameters that are derived based on plant-specific information, such as anchoring and installation of the component or structure and plant-specific material test data. *NOTES: (2) The objective of the fragility analysis is to derive fragility parameters that are as realistic as possible. They should reflect the as-built conditions of the equipment and should use plant-specific information. Use of conservative fragilities would distort the contribution of the seismic events to core damage frequency and large early release frequency. Note that the use of conservative fragilities may underestimate the frequencies of some accident sequences involving "success" terms. Therefore, generic fragilities, if used, should not be overly conservative and should be appropriate for the seismic risk profile of the plant. For further discussion, refer to 5-1.6. Peer reviews need to be attentive to this aspect.*	The IPEEE was reviewed to assure that the SSCs credited in the demonstrably conservative evaluation have fragilities that do not preclude crediting them in the analysis.
SFR-F3 CALCULATE seismic fragilities for relays identified to be essential and that are included in the systems-analysis model. NOTES: (3) Guidance on evaluation of relay chatter effects is given in references [5-3], [5-7], and [5-14] (see	Not applicable.

Requirement SPR-B4). Essential relays are defined in reference [5-14].	
SFR-F4 CALCULATE seismic fragilities for SSCs that are identified in the systems model as playing a role in the large early release frequency part of the seismic PRA. (See Requirements SPR-A1 and SPR-A3.) *NOTES: (4) Generally, the concern is the seismically induced early failure of containment functions. NUREG-1407 [5-7] describes these functions as containment integrity, containment isolation, prevention of bypass functions, and some specific systems depending on the containment design (e.g., igniters, suppression pools, or ice baskets).*	The liner fragility is calculated as documented in calculation 11Q0060-C-028 and is shown to be bounded by the CDT fragilities.
SFR-G1 DOCUMENT the seismic-fragility analysis in a manner that facilitates PRA applications, upgrades, and peer review.	The realistic seismic fragility of the CDTs is calculated as documented in calculations 11Q0060-C-024, -025, -028, -032, & -033.
SFR-G2 DOCUMENT the process used in the seismic-fragility analysis. For example, this typically includes a description of (a) the methodologies used to quantify the seismic fragilities of SSCs, together with key assumptions (b) the SSC fragility values that includes the method of seismic qualification, the dominant failure mode(s), the source of information, and the location of the component (c) the fragility parameter values (i.e., median acceleration capacity, *BETA*(R) and *BETA*(U) and the technical bases for them for each analyzed SSC, and (d) the different elements of seismic-fragility analysis, such as (1) the seismic response analysis (2) the screening steps (3) the walkdown (4) the review of design documents (5) the identification of critical failure modes for each SSC, and (6) the calculation of fragility parameter values for each SSC modeled *NOTE: (1) The documentation requirements given in NUREG-1407 [5-7] and followed in the Diablo Canyon Long Term Seismic Program [5-26] and Bohn and Lambright [5-17] studies may be used as guidance.*	11Q0060-RPT-002, 11Q0060-C-028, and Engineering Evaluation 2017-0008 document the process used in the seismic-fragility analysis, the methodologies, the failure modes, and the fragility parameters.

ATTACHMENT D EPRI FRANX 4.3 HAZARD EDITOR OUTPUT

Acceleration (g)	Exceedance Frequencies 5%	50%	Mean	95%
0.0005	1.57e-02	2.88e-02	2.90e-02	4.19e-02
0.001	1.02e-02	2.35e-02	2.38e-02	3.68e-02
0.005	2.32e-03	7.89e-03	8.51e-03	1.64e-02
0.01	1.04e-03	3.68e-03	4.28e-03	9.79e-03
0.015	6.00e-04	2.10e-03	2.62e-03	6.83e-03
0.03	1.69e-04	6.64e-04	9.36e-04	2.92e-03
0.05	5.35e-05	2.42e-04	3.69e-04	1.20e-03
0.075	1.84e-05	1.01e-04	1.67e-04	5.50e-04
0.1	8.23e-06	5.35e-05	9.41e-05	3.14e-04
0.15	2.60e-06	2.19e-05	4.14e-05	1.42e-04
0.3	3.05e-07	4.37e-06	9.28e-06	3.28e-05
0.5	5.27e-08	1.13e-06	2.66e-06	1.01e-05
0.75	1.11e-08	3.23e-07	8.43e-07	3.37e-06
1	3.42e-09	1.16e-07	3.36e-07	1.40e-06
1.5	5.12e-10	2.35e-08	8.07e-08	3.52e-07
3	9.11e-11	1.02e-09	5.89e-09	2.68e-08
5	7.77e-11	1.36e-10	7.95e-10	3.73e-09
7.5	7.13e-11	9.11e-11	1.44e-10	7.34e-10
10	7.13e-11	9.11e-11	3.93e-11	2.53e-10

OBE must be less than SSE

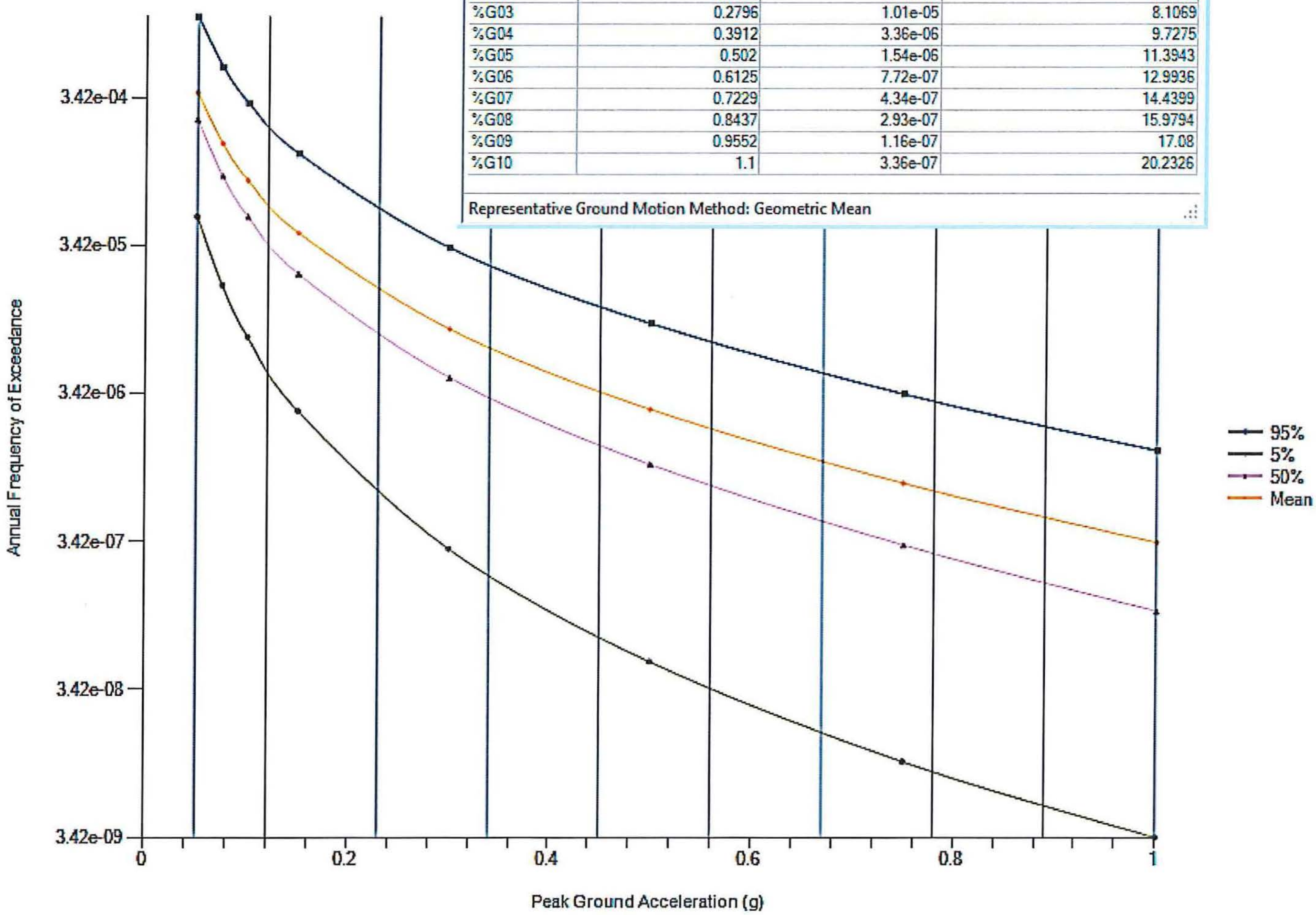
Representative Ground Motion Method: Geometric Mean

Number of Bins: 10

ID	Lower	Upper
%G01	0.05	0.12
%G02	0.12	0.23
%G03	0.23	0.34
%G04	0.34	0.45
%G05	0.45	0.56
%G06	0.56	0.67
%G07	0.67	0.78
%G08	0.78	0.9125
%G09	.89	1
%G10	1	

Bin	Representative Ground Motion (g PGA)	Mean Frequency	Error Factor
%G01	0.0775	3.04e-04	4.4913
%G02	0.1661	4.82e-05	6.1928
%G03	0.2796	1.01e-05	8.1069
%G04	0.3912	3.36e-06	9.7275
%G05	0.502	1.54e-06	11.3943
%G06	0.6125	7.72e-07	12.9936
%G07	0.7229	4.34e-07	14.4399
%G08	0.8437	2.93e-07	15.9794
%G09	0.9552	1.16e-07	17.08
%G10	1.1	3.36e-07	20.2326

Representative Ground Motion Method: Geometric Mean



ENCLOSURE 5

NextEra Energy Point Beach, LLC

License Amendment Request (LAR) 278

Engineering Evaluation 2017-0008

(58 pages follow)

Point Beach Nuclear Plant

Engineering Evaluation 2017-0008

Technical Summary of Methodology and Criteria to Determine the Strength Capacity of the Containment Dome Trusses and Attached/Adjacent Components in Support of License Amendment Request 278

Rev. 0
March 29, 2017

Table of Contents

1.0 Purpose

The purpose of this engineering evaluation is the following:

- To describe the background of the structural nonconformances (i.e. nonconformance to the design codes of record) relating to the construction trusses[1] and attached/adjacent components.

- To describe the technical bases for the methodology and acceptance criteria which are used by the calculations that support the risk assessment developed for the risk-informed license amendment request.

- To summarize the structural evaluations performed in support of a risk-informed license amendment request associated with the trusses, attached components (i.e. supported by the truss structures), and localized adjacent (to the truss structures) areas of the containment liner[2] and structure. These analyses perform two specific functions:

    - The calculations include supplemental evaluations used to demonstrate that the trusses are capable of maintaining structural integrity, i.e., remain capable of supporting applied dead and live loads during and after either a seismic event, based on a ground motion response spectrum (GMRS), or a design basis accident (DBA) while not impeding/impairing the function of supported or adjacent systems, structures, or components (SSCs).

    - These supplemental analyses also provide the foundation to develop the seismic and thermal strength capacity which is directly used in the development of the risk assessment calculations that support the risk-informed license amendment request.

[1] The use of the term truss or trusses throughout this engineering evaluation will refer to the construction truss/trusses. Other calculations, drawings, reports, and legacy documents may refer to the trusses as construction trusses, dome liner erection trusses, containment dome trusses, etc.
[2] The use of the term containment liner or liner used throughout this evaluation will refer to the applicable unit's containment liner.

2.0 Current Licensing Basis

2.1 Seismic Loading

2.1.1 Ground Motion

The source time history/ground motion at Point Beach is documented in Appendix A.5, "Seismic Design Analysis", of the Updated Final Safety Analysis Report (UFSAR) (Ref. 3.1), which states:

> *The spectrum response curves for the equipment inside the building are generated by the time history technique of seismic analysis. The sample earthquake utilized is that recorded at Olympia, Washington 45N-120W on April 13, 1949. The originally recorded earthquake is scaled to that of .06g. Essentially, the curves are generated by applying the recorded earthquake to a single degree of freedom system, for which the values for damping and natural frequency are varied.*

2.1.2 Damping

Table A.5-2, "Damping Factors", of the UFSAR (Ref. 3.1) gives the damping factors used in the design of components and structures.

Type of Condition and Structure	Design Earthquake	Hypothetical Earthquake
Welded Steel Plate Assemblies	1%	2%
Welded Steel Framed Structures	2%	2%
Bolted Steel Framed Structures	2.5%	5%
Interior Concrete Equip. Supports	2%	2%
Reinforced Concrete Structures on Soil	5%	7.5%
Prestressed Concrete Containment Structure on Piles	2%	5%
Vital Piping Systems	0.5%	0.5%
Soil Damping	5%	5%

The Design Earthquake is that produced by a ground motion with a peak horizontal acceleration of 0.06g and peak vertical acceleration of 0.04g. It is alternatively identified as the Operating Basis Earthquake (OBE).

The Hypothetical Earthquake is that produced by a ground motion with a peak horizontal acceleration of 0.12g and peak vertical acceleration of 0.08g. It is alternatively identified as the Safe Shutdown Earthquake (SSE).

2.1.3 Development of Containment In-Structure Response Spectra

The containment building shell and dome consist of (horizontally and vertically) prestressed post-tensioned concrete per Section 5.1.2.1 of the UFSAR (Ref. 3.1). The internal structure is comprised of interconnected reinforced concrete floors and walls and is independent of the shell and dome. The shell and internal structure rest on the same circular reinforced foundation slab, which rests on steel piles driven to bedrock.

The model for the development of the containment in-structure response spectra consists of a stick model of the shell and internal structure with appropriate section properties and mass (Ref. 5.1). The horizontal effects of the soil are represented by a spring. The vertical and rotational effects of the steel piles are represented by two springs located diametrically opposite to each other at the edge of the foundation slab.

Damping values for the structural system were selected based upon evaluation of the materials and mode shapes. The evaluation of the mode shapes made possible the selection of damping values to be associated with each mode. Both the first and the second mode indicated activity of both the structure and soil. The first mode showed the soil to be contributing to translation and some rocking effect of the building and the concrete internals. The second mode showed similar motions as the first mode. The higher modes showed mainly flexure of the structure with some translation in the soil. A conservative value for damping of 2% and 5% was used for all modes respectively for the design and hypothetical earthquakes.

2.2 Thermal Loading

Normal Operation

Per Section 5.1.1.1 of the UFSAR (Ref. 3.1), the normal operating temperature (ambient) inside containment is between 50°F and 120°F.

Design Basis Accident

Per Section 6.3.1 of the UFSAR (Ref. 3.1), the maximum design temperature inside containment after a design basis accident (e.g. loss of coolant, steam line break) is 286°F.

2.3 Acceptance Criteria

2.3.1 Seismic Loading

Appendix A.5, "Seismic Design Analysis", of the UFSAR (Ref. 3.1) states:

> All components, systems, and structures classified as Class I are designed in accordance with the following criteria:
>
> 1. Primary steady state stresses, when combined with the seismic stresses resulting from a response spectrum normalized to a maximum ground acceleration of 0.04g in the vertical direction and 0.06g in the horizontal direction simultaneously, are maintained within the allowable stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards, e.g., ASME Boiler and Pressure Vessel Code, USAS B31.1 Code for Pressure Piping, ACI 318 Building Code Requirements for Reinforced Concrete, and AISC Specifications for the Design and Erection of Structural Steel for Buildings.
>
> 2. Primary steady state stresses when combined with the seismic stress resulting from a response spectrum normalized to a maximum ground acceleration of 0.08g acting in the vertical direction and 0.12g acting in the horizontal direction simultaneously, are limited so that the function of the component, system or structure shall not be impaired as to prevent a safe and orderly shutdown of the plant.

Station practice (Ref. 3.3) for acceptance criteria for the dead load and SSE loading condition is to limit allowable stress values to 1.5 times the acceptance criteria of Ref. 2.1, not to exceed 90% of the material yield strength.

As identified above and in the Point Beach UFSAR, the design code of record for the following SSCs is:

- Trusses – AISC Manual of Steel Construction, 6th Ed.
- Containment Spray – USAS B31.1 Power Piping, 1967

- Post-Accident Containment Ventilation (PACV) Piping – USAS B31.1 Power Piping, 1967
- Containment Liner – Discussed in Section 2.3.3
- Containment Structure – ACI 318-63
- Containment Air Recirculation Cooling System (VNCC) Ductwork – Ref. 3.16 identifies a design guideline, but no structural guidance is available, use AISC 6th Ed. in accordance with Ref. 3.2
- Lighting and other Misc. Loads – Assessed on a case by case basis

2.3.2 Thermal Loading

The UFSAR (Ref. 3.1) is silent on specific acceptance criteria for thermal loading for steel structures. Station practice for acceptance criteria for the dead load and thermal loading condition is per Ref. 3.2, which limits it to 1.33 times the acceptance criteria of Ref. 2.1, not to exceed 90% of the material yield strength.

2.3.3 Liner Plate

Section 5.1.2.2 of the UFSAR (Ref. 3.1) states:

> The following sections of the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels, Article 4, are used as guides in establishing allowable strain limits:
>
> 1. Paragraph N-412(m)
> 2. Paragraph N-414.5
> 3. Table N-413
> 4. Figure N-414, N-415(A)
> 5. Paragraph N-412(n)
> 6. Paragraph N-415.1
>
> Implementation of the ASME design criteria requires that the liner material be prevented from experiencing significant distortion due to thermal load and that the stresses be considered from a fatigue standpoint. [Paragraph N-412(m)(2)]
>
> ...
>
> The maximum compressive strains are caused by accident pressure, thermal loading, prestress, shrinkage, and creep. The maximum strains do not exceed 0.0025 in./in. and the liner plate always remains in a stable condition.

Per Section 5.1.1.5 of the UFSAR (Ref. 3.1), the applicable ASME Code is the 1965 edition.

2.4 Material Properties

Material strengths used for the truss, liner, piping, and ductwork are be based on published minimum strength values. Use of minimum strength values combined with code design allowable stress values provide additional design margin that offsets the effect of uncertainty. Piping allowable stress values are taken per USAS B 31.1., 1967 Ed. Containment concrete strength is as discussed in Section 6.3. Uncertainty associated with the seismic fragility is addressed in Section 6.4.2. The thermal risk discussed in Section 6.4.1 is developed using code allowable values and minimum strength values, and therefore is considered to include adequate margin to accommodate uncertainty.

Below is a summary of material properties:

- Dome Truss – A-36 carbon steel, as noted in the original specification (Ref. 3.13, §15.3.1), structural shapes for supports per UFSAR §5.6.1.7, and shop drawings (Ref. 4.2)
- Containment Liner – ¼" thick ASTM A442, Grade 60 per Ref. 3.4 and UFSAR §5.6.1.7
- Containment Spray – A312 Type 304 Stainless Steel, per UFSAR Table 6.4-5
- PACV piping – A312 Type 304 Stainless Steel, per Ref. 3.4
- Ductwork – Sheet metal/steel per Ref. 3.16
- Lights/Conduit/Misc. – Addressed on a case by case basis, as necessary
- Containment Structure – 5000 psi concrete at 28 days per UFSAR §5.6.1.7 (further requirements for the concrete ingredients are provided in the UFSAR)

3.0 Background

3.1 Truss Original and Current Purpose

The trusses (Ref. 4.1) were installed during the original construction of the plant. There is one truss structure in each of the two containment units. The original purpose of the trusses was to support the dome roof steel liner and the first lift of wet concrete over the liner. The first lift of concrete varied in thickness from 12" over the second top chord segment near each support to 8" over the remainder of the truss (Ref. 4.1). Subsequent to the hardening of the first lift of concrete, the truss was lowered by approximately 3" at the support brackets to isolate the truss from the containment liner and left in this lowered position (Ref. 3.7). The trusses were subsequently used to support the containment spray ring headers (Ref. 4.3), the PACV system (Ref. 4.4), a portion of the VNCC ductwork (Ref. 4.5), lights, and miscellaneous conduit.

3.2 Truss Configuration

3.2.1 Construction (Ref. 4.1, 4.2)

The trusses are carbon steel structures (specified as ASTM A36 per §15.3.1 of Ref. 3.13). Each dome truss consists of eighteen (18) equally spaced (in a circular pattern) vertical trusses. The top chord follows the dome contour. The bottom chord starts at a common point with the top chord then follows the top chord but diverges to end at approximately 8 ft. below the end of the top chord (Ref. 4.1).

The top and bottom chords segments are WT shapes (the original drawings note the members as ST, this was the convention used at the time of construction and is equivalent to a WT today). The segments are connected to each other with full penetration welds. Vertical bracing connects the top and bottom chords. The vertical bracing consists of double angles and are welded to the web of the WT sections.

The trusses are divided into two configurations: six (6) trusses labeled as T1 and twelve (12) trusses labeled as T2 (Ref. 4.1). T1 trusses are fabricated with heavier WT and double angle shapes than the T2 trusses. Two T2 trusses are positioned between adjacent T1 trusses. The T1 trusses are bolted at the apex to a built-up hexagon-shaped plate frame. The T2 trusses are bolted to a hexagon-shaped built-up beam system, which is bolted to the top chord, and located at approximately 7'-11" from the center of the dome truss (Ref. 4.1).

The top chords are connected to each other with 8WF beams that are bolted to tabs that are welded to the WT sections. The bottom chords are connected to each other in every other bay with horizontal double angle bracing that is bolted to gusset plates that are welded to the WT sections. The top chords are connected to the adjacent bottom chords in every other bay with double angle vertical (diagonal) bracing that is bolted to gusset plates that are welded to the WT sections.

The base of each top and bottom chord is welded to a horizontal plate to the bottom of which a rocker (milled curved block of steel, see Detail 2 of Ref. 4.1) is welded. The rockers allow each truss to pivot at that location. The rocker rests in a rocker pocket, which is on top of a bearing box, which fits into a bearing housing (i.e., the truss bears on the top of the bearing box under self-weight with no other physical connection to the bearing box). The rocker pocket, bearing box, and bearing housing are fabricated from welded plate sections (see Ref. 4.1 for additional details).

The bearing housing is welded to a baseplate that is bolted to a support beam. The top of each support beam is at elevation 124'-5¼" (Ref. 4.1). The bolt holes in the baseplate are slotted in the radial direction. Two ¼" thick graphite plates are placed between the baseplate and the support beam. The slotted bolt holes and graphite plates allow each truss to move radially. The bolts bear on the long side of the slotted holes and provide lateral (tangential) restraint to the trusses. The support beams are embedded in the containment building concrete wall. The baseplates (of the truss with attached rocker) are connected with horizontal rods to a horizontal plate at the center of the dome truss.

3.2.2 Proximity to Liner (Ref. 3.11, 3.12)

As depicted in Ref. 4.1, the truss placement is concentric with the containment structure, which suggests that there is equal clearance between the truss and the liner. Using dimensions from Ref. 4.1 and 4.2, the calculated clearance at each of the 18 top chord locations is 1.09" (Ref. 5.8). However, the as-found clearance between the 18 top chords first panel points and the liner varies from 0" to 1.75" for Unit 1 (Ref. 3.11) and from 0.5" to 1.9375" for Unit 2 (Ref. 3.12).

3.3 Truss Analysis – Current Licensing Basis

Each truss is subject to the following loads:

- Dead load due to self-weight of truss and attached components (containment spray lines - both empty and filled with water, PACV lines, lights, and VNCC ductwork).
- Seismic inertia loads due to self-weight.
- Seismic loads from attached components.
- Thermal loads due to differential thermal displacements between the truss and the containment spray lines during a design basis accident inside containment.
- Thermal loads due to the insufficient clearance to account for truss thermal expansion.

Per Appendix A.5, "Seismic Design Analysis", of the UFSAR (Ref. 3.1) the seismic loads are the maximum loads due to one horizontal (either direction) earthquake combined with the vertical loads due to the earthquake.

UFSAR Section 5.1.2.2 states that the containment structure is designed for the following loading cases:

1. $D + F + L + T_o$
2. $D + F + L + P + T_A + W$ (or E)
3. $D + F + L + P'$

where: D = Dead Load
L = Appropriate Live Load
F = Appropriate Prestressing Load
P = Pressure Load (Varies with Time from Design Pressure to Zero Pressure)
T_o = Thermal Loads Due to Operating Temperature
T_A = Thermal Loads Based on a Temperature Corresponding to a Pressure P
E = Design Earthquake Load
P' = Test Pressure (1.15 P)
W = Wind Load

Even though the truss is supported by the containment structure it provides no structural support/reinforcement to the containment structure, nor does it contribute to the ability of containment to perform its intended design basis functions. Therefore, the truss is considered a separate structure and the Design Earthquake Load and Design Basis Accident load combination requirements (load combination 2 above) for the trusses and attached components (except the containment structure as noted above) are not applicable.

3.3.1 Seismic Analysis

A requirement for leaving the dome truss in place (during original construction) was to evaluate it for seismic loads (Ref. 3.8). As no original seismic evaluation of the trusses was located, a seismic analysis of the trusses was performed in 1990 (Ref. 5.2), with the effect of attached components to be evaluated at a later date. The results of the analysis in Ref. 5.2 were also used to validate the application of the seismic response spectra of the containment wall for the seismic analysis of the containment spray lines that are attached to the trusses (Ref. 3.15).

The analyzed truss was that corresponding to the configuration depicted in Ref. 4.1 (note that Ref. 4.1 was at revision level 6 at the time when the 1990 analysis was performed). The analysis used current licensing basis seismic criteria. The analysis was for dead load and Operating Basis Earthquake (OBE) and for dead load and Safe Shutdown Earthquake (SSE). The seismic accelerations were obtained from the response spectra at elevation 105' of the containment building. It was determined in Ref. 5.2 that the dynamic horizontal frequencies of the truss ranged from 7 Hz to 9.74 Hz. Because the corresponding accelerations in this frequency range are near the zero period acceleration (ZPA) of the response spectra, a static analysis of the truss was performed.

The analysis results showed that the stresses in the truss components met the current licensing basis acceptance criteria for structural steel.

During the 2011 NRC Component Design Basis Inspection (CDBI), a question was raised (Ref. 3.14) relative to an open item in the containment spray piping calculations regarding the analysis of the trusses for the applied containment spray pipe support loads. The open item indicated that the evaluation of the trusses was outside the scope of the piping analysis.

The truss was reanalyzed (Ref. 5.4) for seismic loading in accordance with the current licensing basis seismic criteria and using the acceleration response spectra at elevation 125 ft. The weights of attached components were included in the inertia loads. Containment spray and PACV pipe support loads were also applied. During a review of truss photos and shop drawings (Ref. 4.2), it was observed that the outermost horizontal braces intersected the bottom chords at approximately 2'-5" from the support point, instead of at the support point as depicted in Ref. 4.1 and used in Ref. 5.2. Henceforth, the truss model was updated for the analysis of this configuration and for other boundary condition updates including permitting radial movement at all truss support locations.

The analysis performed in Ref. 5.4 revealed that in the as-built configuration, the trusses are significantly more flexible than previously analyzed (Ref. 5.2); therefore, the trusses would be subjected, during a seismic event, to accelerations that are higher than those used in Ref. 5.2. The results of the revised analysis showed that stresses in the top and bottom chord segments exceeded the material yield strength, specifically at the intersection of the outermost horizontal braces with the bottom chords. Furthermore, stresses in the bottom chords at containment spray line anchor locations exceeded the material yield strength.

In an effort to show that the seismic demand used in the analysis performed in Ref. 5.4, as obtained from Ref. 5.1, was conservative, the seismic demand was refined with the use of current day soil-structure interaction techniques (Ref. 5.5), and the truss reanalyzed as documented in Ref. 5.6.

The proximity of the trusses to the liner results in contact with the liner during a seismic event. This contact imposes a local load on the liner, which results in local liner stresses.

An operability evaluation of the trusses and the liner were performed. It was concluded that the trusses did not conform to the design code of record and the containment liner/structure were operable but non-conforming (see Section 3.4.3 for containment spray piping). The operability evaluation was reviewed by the NRC (Ref. 1.19).

3.3.2 Thermal Analysis

The truss mounting bolt holes are slotted radially, which permits thermal expansion (radial) of the truss due to a temperature change within containment. The ability to freely expand in the radial direction precludes the development of thermal loading in the trusses. However, the trusses will be subject to thermal loading as a result of differential thermal displacements between the truss and the containment spray lines during a design basis accident inside containment.

The as-found clearances between the truss and the liner identified in Ref. 3.11 and 3.12 result in contact between the truss and the liner during thermal expansion from a DBA. This contact imposes additional loads on the truss that results in stresses in truss components exceeding the material yield strength. Additionally, this contact imposes a local load on the liner, which results in localized containment liner/structure stresses.

The trusses and attached/adjacent components were analyzed in support of an operability evaluation for the thermal loading as a result of a DBA. The analysis accounted for the reduced clearances with the liner by modeling non-linear springs to account for the gap and the liner contact stiffness. The analysis results are summarized as follows:

- Contact loads with the liner were less than the liner contact capacity.
- Stresses in a select number of truss bottom chords exceeded the material yield strength but the sections did not develop a plastic hinge.
- A plastic hinge was formed in one Unit 1 truss bottom chord but the trusses were shown to maintain structural integrity.
- The loads in a select number of Unit 1 truss Detail 4 (Ref. 4.1) clip angle connections exceeded the material yield capacity but the connection was shown to maintain the structural integrity of the truss.
- Attached components were determined to maintain structural integrity.

It was concluded that the trusses did not conform to design code of record values, and the containment - liner/structure were operable but non-conforming, and the attached components were fully operable (for thermal loading). The operability evaluation was reviewed by the NRC (Ref. 1.19).

3.4 Containment Spray Pipe Stress Analysis – Current Licensing Basis

Per Ref. 3.10, seismic loads are not postulated to be concurrent with a design basis accident inside containment, and the pipe stress analyses in Ref. 5.25 and 5.26 have evaluated the seismic and thermal design basis accident loads separately.

3.4.1 Description

There are two (2) containment spray lines in each containment building. Starting at the penetration, each line runs vertically along the containment wall then crosses over towards the center of the containment and forms a loop (Ref. 4.2). The outer loop is anchored at four (4) locations (Ref. 4.3.2 and 4.3.4) to the bottom of the truss bottom chords, and supported vertically with trapeze supports that are attached to the bottom of the remainder of the bottom chords. The inner loop is anchored at three (3) locations (Ref. 4.3.1 and 4.3.3) to the bottom of the truss bottom chords, and supported vertically with trapeze supports that are attached to the bottom of the remainder of the bottom chords. The penetrations at the containment building wall are also anchors.

3.4.2 Acceptance Criteria

The original code of record for the containment spray piping is USAS B31.1 Power Piping Code, 1967 edition (Ref. 2.3). Per Ref. 3.5, ASME B&PV Code, Section III, subsections NC and ND, 1977 edition up to and including 1978 Winter addenda (Ref. 2.4) has been reconciled with Ref. 2.3 to be used for pipe stress analyses, with values for the allowable stresses to be taken from USAS B31.1 Power Piping Code, 1967 edition. The supports are evaluated in accordance with Ref. 3.6.

3.4.3 Seismic Analysis

The containment spray lines were originally analyzed (Ref. 5.25 & 5.26) using the 0.5% damped acceleration response spectra of the containment building elevation 105 ft. Since the conclusion from (Ref. 5.2) was that the truss dynamic response was near the ZPA of the response spectra, the response spectra at elevation 105' was also applied to the lines that are attached to the trusses, and consequently, no relative displacements between the lines attached to the truss bottom chords and the lines attached to the containment wall were applied.

The analysis performed in (Ref. 5.4) revealed that in the as-built configuration the trusses are significantly more flexible than previously analyzed (Ref. 5.2), which amplifies the seismic response of the truss, and therefore, during a seismic event subjects the containment spray lines to higher accelerations as well as differential displacements with the containment wall.

The containment spray lines were reanalyzed in Ref. 5.4 for the amplified seismic response and for the relative displacements of the trusses with the containment wall. The results showed that stresses in the pipe and supports exceeded the acceptance criteria.

An operability evaluation of the containment spray lines was performed (Ref. 5.4) in accordance with Ref. 3.5, 3.6. It demonstrated that the containment spray pipe and supports did not conform to the design code of record, however, the analysis concluded they were operable but non-conforming. The operability evaluation was reviewed by the NRC (Ref. 1.19).

3.4.4 Thermal Analysis

The containment spray lines are analyzed (Ref. 5.25 & 5.26) for the following three thermal loading conditions:

- The truss and empty pipe are both at the same elevated temperature after a design basis accident inside containment. The loading in the line is developed from the differential displacement resulting from the difference in coefficients of thermal expansion between the truss carbon steel and the containment spray pipe stainless steel material.

- The truss is at the elevated temperature after a design basis accident inside containment, while the cold filled pipe is at 70°F when the containment spray system is activated.

- The truss is at the elevated temperature after a design basis accident inside containment, while the hot filled pipe is at 206°F when the containment spray system is activated. The loading condition is bounded by the cold filled pipe loading condition.

An evaluation was performed to assess the effect of the as-found clearances between the trusses and the liner on the containment spray lines. It was concluded that the existing analyses (Ref. Ref. 5.25 & 5.26) were acceptable. The evaluation was reviewed by the NRC (Ref. 1.19).

3.5 Resolution of Trusses and Containment Spray Lines Structural Nonconformances

Resolution of the trusses and the containment spray lines structural nonconformances to meet current licensing basis would require the implementation of various modifications. Examples of these modifications are:

- Trim truss top chords (trimming ranges from removal of material at flange tips to complete removal and replacement of the flange) at discrete locations to prevent contact with the liner.
- Modify truss support conditions (to reinforce and limit total movement of the bearing housing).
- Strengthen and stiffen top and bottom chords.
- Modify several containment spray pipe supports.

The trusses are located in the containment buildings, approximately 60 to 90 feet above the operating floor elevations. Rigging and handling of large structural members at extreme elevations with limited safe access points could present an unnecessary risk to the station from a potential dropped object, or unnecessary risk to worker safety.

The risk-informed resolution limits the scope of modifications to resolve and accept the low risk structural nonconformances related to a legacy condition, while demonstrating that safety margins are maintained.

Section 4.0 of this evaluation contains the methodologies to determine the strength capacities, the seismic fragilities, and the thermal risks, to be used for the risk assessment of the trusses and attached/adjacent components (with limited scope modifications). Section 4.0 of this evaluation also contains the methodology and acceptance criteria which are used in additional calculations that support the conclusion of the risk assessment by demonstrating that the trusses and attached/adjacent components (with limited scope modifications) maintain structural integrity for required thermal and seismic loads. Section 5.0 contains the technical evaluation. Section 6.0 contains a discussion on the analyses that were performed to support the risk-informed approach and contains a summary of the results of each analysis.

4.0 Methodology and Criteria to Support the Risk-Informed LAR

The methodologies to determine the strength capacities and the methodologies to develop the seismic fragilities and the thermal risks, to be used for the risk assessment of the trusses and attached/adjacent components (with limited scope modifications), are discussed in Ref. 6.9.

Ref. 6.9 also discusses the methodology and acceptance criteria which are used in additional calculations that support the conclusion of the risk assessment by demonstrating that the trusses and attached/adjacent components (with limited scope modifications) maintain structural integrity for required thermal and seismic loads.

The general outline of the analyses, which are summarized in Section 6.0, to support the risk-informed approach, is as follows:

- Develop the liner contact capacity.
- Seismic Analysis
  - Develop site-specific response spectra based on the site-specific hazard.
  - Develop the in-structure seismic demand based on a soil-structure interaction (SSI) analysis.
  - Analyze the trusses and attached/adjacent components (existing configurations) to determine the most limiting components.
  - Determine the seismic fragility of the trusses and attached/adjacent components (existing configuration).
  - Analyze the trusses and attached/adjacent components (modified configuration/no liner contact) to determine the most limiting components.
  - Determine the seismic fragility of the trusses and attached/adjacent components (modified configuration/no liner contact).
  - Analyze the trusses and attached/adjacent components (existing configurations) for the in-structure seismic demand to assess structural integrity.
- Thermal analysis
  - Identify the probability of failure vs temperature of the trusses (existing and modified configurations, if required) under thermal loads as a result of a DBA.
  - Analyze the trusses (existing configuration) for thermal loads as a result of a DBA to assess structural integrity.
  - Analyze the trusses (modified configuration, if required) for thermal loads as a result of a DBA to assess structural integrity.
- Identify seismic and thermal limits for events of lesser magnitudes under which the stress levels for truss and attached components (modified configuration, if required) are within the elastic range.

Tables 5-1 through 5-3 summarize the methodologies and acceptance criteria that are discussed in Ref. 6.9 and that are used for the above analyses. Below is a brief overview, except for the methodology for determining seismic fragility and probability of failure vs temperature which are discussed in Section 5.0, discussing the technical evaluation of the methodologies and acceptance criteria.

4.1 Seismic and DBA Loads

As discussed in Section 3.3, the truss is not integral with the containment structure, nor does it provide a structural containment support function. The load combinations applicable to the containment structure per UFSAR Section 5.1.2.2 are not required for the truss. Therefore, the methodology and acceptance criteria state that seismic loading and design basis accident thermal loading for the trusses and attached components are not concurrent. This does not apply to the containment liner since it is considered part of the containment structure system.

4.2 Truss and Attached Components

In an effort to determine seismic fragility values and assess structural integrity, the trusses are evaluated against criteria from ASCE/SEI 43-05 for Limit State D, "Essentially Elastic", within the limitations of NUREG/CR-6926 (Ref. 1.17). ASCE/SEI 43-05 is being selected since it covers the following essential aspects of seismic evaluation:

- Ground motion input
- In-structure response spectra
- Damping
- Structural analysis
- Acceptance limits

For consistency, the same code used to evaluate truss components under seismic loads are used for the DBA thermal loads.

If the acceptance limits are exceeded for the truss top or bottom chords, strain acceptance criteria are used.

The code guidance used to evaluate the attached components remains the same as the current design basis guidance (i.e. USAS B31.1 1967 Ed., etc.). The seismic input to attached components follows the seismic input developed as discussed in Sections 6.2 and 6.6.2.

4.3 Liner and Containment Wall

The methodology and criteria to support the capacity/fragility analysis with respect to the liner is stress-based in place of strain-based. The methodology and criteria also accounts for the repeated seismic contact load with the liner. The methodology and criteria allows localized concrete strain limit exceedance as long as liner leak-tight integrity is maintained, and uses concrete compressive strength based on the compressive strength from test data. See Table 4-3.

Table 4-1: Summary of Methodology and Acceptance Criteria to Demonstrate Structural Integrity		
Item	**Current Criteria**	**Alternative Criteria**
Ground seismic input	SSE is based on a Housner spectral shape ground response spectra (GRS) anchored to a 0.12g PGA. Vertical accelerations are taken as 2/3 of the horizontal GRS value.	Apply a ground motion seismic based on a site-specific GMRS. Apply EPRI 3002004396 Appendix A to define the vertical spectral shape using mean V/H ratios for A-Soft sites. See Table 4-2 for methodology to determine seismic demand at the truss supports.
Acceptance criteria for truss components subject to seismic loads	As discussed in Section 2.3.1.	Analyze for GMRS-based seismic load. Apply seismic analysis and acceptance criteria from ASCE/SEI 43-05, Limit State D to meet AISC N690-1994(R2004)[3], with stress increase factor from Table Q1.5.7.1. Exception: If AISC N690-1994 acceptance criteria are not met for the truss top and/or bottom chords, limit the maximum strain to 1.5% for combined axial and flexure or flexure only.
Acceptance criteria for attached components subject to seismic loads	As discussed in Section 2.3.1 and 3.4.2.	For system segments attached to and significantly influenced by the truss motion, evaluate seismic load combinations only the using GMRS-based seismic load. No change in acceptance criteria.
Steel liner under contact load from truss	No defined criteria, liner contact load not considered for original design.	See Table 4-3.
Containment structure concrete behind liner at contact point with truss	No defined criteria, liner contact load not considered for original design.	See Table 4-3.
Acceptance criteria for truss components subject to DBA thermal loads	As discussed in Section 2.3.2.	Apply N690 strength criteria, with stress increase factor from Table Q1.5.7.1.

[3] Where AISC N690 is referred to in this document, it is in reference to AISC N690-1994(R2004).

Table 4-2: Summary of Seismic Analysis Criteria and Methods		
Item	**Current Criteria/Method**	**Criteria/Method for Risk-Informed Approach**
In-structure seismic motion at truss support locations	Determined through SSI analysis with simplified springs to represent soil effects.	SSI analysis state-of-practice analysis methods. See Section 6.2.
Ground motion time history for SSI analysis	Olympia, Washington 45N-120W (N80E) on April 13, 1949, scaled to 0.06g for OBE. SSE is two times OBE.	Time histories to meet ASCE/SEI 43-05 Section 2.4 with limitations identified in NUREG/CR-6926. See Section 6.2.1.
Soil properties for SSI analysis	Simplified springs to represent soil effects.	As determined by SSI analysis for site soil profile. See Section 6.2.
Structural damping of the prestressed concrete containment structure in SSI analysis	5%	5% (No change from current licensing basis – shown here for completeness)
Truss damping for seismic response analysis	5%	7% for bolted steel structure. See Section 5.3.
Damping for attached piping	0.5%	4% See Section 5.3

Table 4-3: Summary of Liner Criteria for Seismic or DBA Loads	
Item	**Criteria/Method**
Steel liner at truss contact point	Allowable load under seismic of DBA loads is the minimum of: • The load that develops a maximum primary stress intensity of 0.9Su • 2/3 of the maximum sustainable load
Steel liner at truss contact point due to repeated contact load	Apply loading/unloading cycles. With each loading cycle, use the displaced shape of the liner from the previous cycle. Determine the accumulation in strains in the liner, and the change in strain between each loading cycle, to assess liner integrity in combination with the fatigue curve from Figure I-9.1 of Ref. 2.5.
Containment structure concrete at truss contact points	Permit localized exceedance of permissible concrete strain allowable limit of 0.003 in/in per ACI 318-63 (Ref. 2.2). Concrete compressive strength based on the compressive strength from test data as permitted in ACI 318-63.

5.0 Technical Evaluation of Methodology and Criteria

NOTE: Criteria obtained from NRC NUREGs or Regulatory Guides throughout this report are used as guidance and do not imply a licensing commitment to the complete document.

Below is a technical evaluation of the methodology and criteria that is summarized in Section 4.0 and discussed in Ref. 6.9.

5.1 Use of Site-Specific GMRS

The use of the site-specific GMRS is consistent with guidance provided in SRP 3.7.1 (Ref. 1.1) that indicates design response spectra are typically developed for site-specific ground motion response spectra. The use of the GMRS in development of the response spectra used for evaluation of the trusses is appropriate for the risk-informed submittal, as it fully reflects the site-specific hazard and is appropriate for use with ASCE/SEI 43-05. While aspects of the Point Beach original design basis ground response spectra, GRS, (scaled Housner curve) reflect some site-specific aspects, including anchoring to site specific peak ground acceleration, it does not reflect all of the revised and updated hazard analyses that are captured within the GMRS. As noted in ASME RA-Sa-2009 (Ref. 2.11), some of the supporting requirements necessary for the risk assessment are based on current data, and use of the GMRS is the most appropriate input response spectra to use to satisfy and support the risk assessment.

The use of the GMRS is appropriate because it is smooth and broad and reflects the site-specific hazard. Also, it is appropriate to use the GMRS in combination with the seismic analysis methods of ASCE/SEI 43-05, as specified in Section 2 of ASCE/SEI 43-05 and as discussed in Regulatory Guide 1.208, which cites the methodology of ASCE/SEI 43-05 to develop performance-based site-specific earthquake ground motion.

The development of the ground motion time histories that match the GMRS, and to be used in the soil-structure interaction analysis, is per Section 2.4 of ASCE/SEI 43-05 with the limitations identified in NUREG/CR-6926. This is consistent with NUREG-0800, Standard Review Plan (SRP) Section 3.7.1 (Ref. 1.1), Acceptance Criterion 1B, Option 2, which requires that (a) through (d) of Option 1, Approach 2, are met (a comparison of requirements is provided in Ref. 6.9 and the more limiting criteria is invoked).

5.2 Development of In-Structure Seismic Response

In-structure response spectra developed through a soil-structure interaction (SSI) analysis is in compliance with SRP 3.7.2 (Ref. 1.2). Even though Section 3.0 of ASCE/SEI 43-05 states that the SSI analysis should follow ASCE 4-98, NUREG/CR-6926 states that some provisions of ASCE 4-98 do not agree with current NRC regulatory documents and staff positions, therefore, the SSI analysis is performed consistent with SRP 3.7.2. The in-structure response spectra are used to determine the seismic fragility of the trusses and to assess structural integrity.

5.3 Structural and Component Damping

Damping is per Table 3-2 of ASCE/SEI 43-05, Response Level 2, which is associated with Limit State D per Table 3-4 of ASCE/SEI 43-05, with the limitations identified in NUREG/CR-6926 (for this application, the limitation applies to piping systems where 4% damping must be used, consistent with RG 1.61, in place of 5% as identified in Table 3-2 of ASCE/SEI 43-05). These damping values are consistent with is RG 1.61. This is also consistent with SRP 3.7.1 and SRP 3.7.2, both of which require that structural and component damping be in accordance with RG 1.61. The following are the damping values that were used for the structural components in the SSI analysis, for the truss analyses, and for the containment spray pipe stress analyses.

Type of Condition and Structure	Damping
Bolted Steel with Bearing Connections	7%
Prestressed Concrete	5%
Piping Systems	4%

Even though each of the individual 18 trusses is a welded planar truss assembly as discussed in Section 3.2.1, the transfer of load between the 18 trusses is through a bolted brace system, and the transfer of load from the containment structure to each truss support is through a bolted connection, therefore, the use of 7% damping is appropriate.

The above damping values from RG 1.61 are for SSE levels. Section 1.2 of RG 1.61 states that SSE damping values are applicable during linear dynamic analysis for in-structure response spectra generation and are based on the expectation that the structural response attributed to load combinations that include SSE will be close to applicable code stress limits. As discussed in Section 6.5.2, under GMRS-based seismic loads, the stress levels in a limited number of the most highly loaded parts of the truss exceed code stress limits. Therefore, using the above damping values is justified.

5.4 Criteria for Truss Components

The structural criteria are per ASCE/SEI 43-05 for Limit State D, "Essentially Elastic", within the limitations of NUREG/CR-6926 (Ref. 1.17). Table 1-4 of ASCE/SEI 43-05 defines "Essentially Elastic" as the limit where no damage occurs. ASCE/SEI 43-05 identifies a set of seismic analysis parameters and design code options associated with this limit state. Sections 1.2 and 4.2.4 of ASCE/SEI 43-05 identify AISC N690-1994 as the Code for structural steel, which is used for the evaluation of truss components using stress increase factors from Table Q1.5.7.1. Section C.I.3.8.4.5 of Regulatory Guide 1.206 (Ref. 1.7) endorsed the use of AISC N690 as the acceptance criteria for seismic category 1 structures.

ASCE/SEI 43-05 has been evaluated for application to nuclear power plants and this evaluation is reported in NUREG/CR-6926. The conclusions of that report support the application of ASCE/SEI 43-05 subject to the limitations stated in the report. As stated in the NUREG/CR-6926 abstract:

> *The overall conclusion from this review effort is that with properly stipulated Performance Goals and supporting criteria, the approach presented in ASCE/SEI Standard 43-05 can provide acceptable levels of protection against severe low-probability earthquakes.*

The primary limitation of NUREG/CR-6926 is "*The Seismic Design Basis for nuclear power plant design and construction should be stipulated as SDB-5D, with a Target Performance Goal (limit state probability) of 10^{-5}/yr*". This limitation is addressed by application of Limit State D (see Tables 1-1 and 1-2 of ASCE/SEI 43-05) in combination with application of the GMRS.

Section 4.2.4 of ASCE/SEI 43-05 states: "AISC N690 shall be modified by the ANSI/AISC 341-02 Provisions, where appropriate". ANSI/AISC 341-02 (Ref. 2.8), modifies the AISC Specification for Structural Steel Buildings to include provisions specific to seismic loading. It is organized in three sections: Part I – Load and Resistance Factor Design (LRFD) design and construction of structural steel buildings, Part II - design and construction of composite structural steel/reinforced concrete buildings, and Part III – Allowable Stress Design (ASD) and construction of structural steel buildings.

The trusses are analyzed per ANSI/AISC N690, which follows the ASD Specification for Structural Steel Buildings, therefore, Part III of ANSI/AISC 341-02 is applicable. However, it is concluded from a review of ANSI/AISC 341-02 that the additional provisions do not apply as discussed below:

- Part III of ANSI/AISC 341-02 modifies the LRFD provisions from Part I of ANSI/AISC 341-02 to include ASD-specific resistance factors. Part III also modifies Section A5.2 of the ASD Specification for

Structural Steel Buildings to increase normal allowable stresses by a factor of 1.7 and modifies Section H1 to remove the factor of safety on the Euler stress used in the combined stress equation. Additional provisions are given for special moment frames, special truss moment frames, special concentrically braced frames, and ordinary concentrically braced frames.

ASCE/SEI 43-05 states "stress limit coefficients in AISC N690 and its Supplement No. 1, appropriate for load combinations that include the Safe Shutdown Earthquake (SSE), can be used to scale the AISC/ASD-allowable stresses to LRFD strength based nominal code capacities". If ASCE/SEI 43-05 intended to apply the 1.7 load factor with the reduction factors from ANSI/AISC 341-02, it would not have referenced the stress limit coefficients in AISC N690. Therefore, the ANSI/AISC N690 stress limit coefficients are used.

Per ANSI/AISC 341-02, special moment frames, special truss moment frames, and special concentrically braced frames are expected to experience significant inelastic deformation during large seismic events, additionally, ordinary concentrically braced frames are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the Design Earthquake. Since Limit State D (essentially elastic) limitations are being applied to the CDT, the additional provisions to the aforementioned frames do not apply.

- Section 6.0 of ASCE/SEI 43-05 identifies ductile detailing requirements where inelastic deformation is allowed. Since Limit State D (essentially elastic) limitations are being applied to the trusses, ductile detailing requirements do not apply.

If, during the assessment of the trusses for structural integrity, the acceptance criteria of AISC N690 cannot be met for the top and/or bottom chords in combined axial and flexure or flexure only, strain acceptance criteria are used to assess the chords. See Section 5.5 below for discussion. This is an exception to AISC N690.

5.5 Strain Acceptance Criteria for Truss Top and Bottom Chords

For design and analysis procedures, SRP Section 3.8.4.I.4 (Ref. 1.4) states that AISC N690, including Supplement 2 (2004), is acceptable for steel structures, and SRP Section 3.8.4.II.5 cites the same for structural acceptance criteria. Furthermore, SRP 3.8.4.III.5 permits those stress limits to be exceeded for some of the load combinations and at some localized points on the structure, with acceptable justification provided to show that the structural integrity will not be affected.

Strain-based acceptance criteria for the truss top and bottom chords is used to account for the significant ductility present in steel components subject to varied loading conditions (Ref. 6.10 & 6.13).

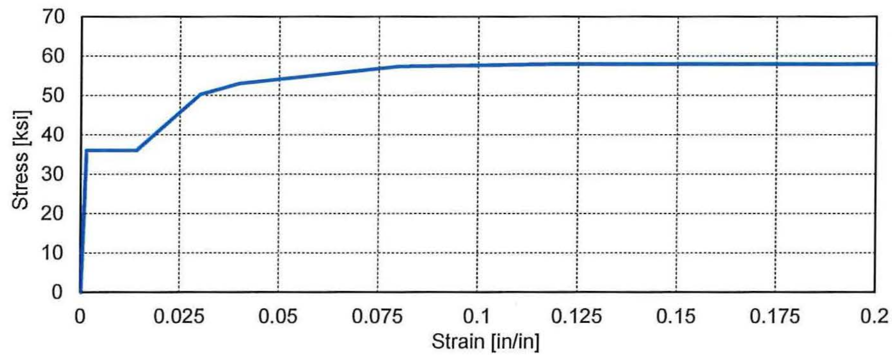
For ASTM A36 steel, Reference 2.9 specifies a minimum yield strength of 36ksi, a tensile strength ranging from 58ksi to 80ksi and a minimum elongation requirement of 20% for an 8" coupon. Reference 6.10 provides stress-strain data (engineering stress-strain representation) for A36 steel from tests. Note that the plotted stress-strain curve in Reference 6.10 is adjusted to minimum specified material yield and tensile strengths which provides a conservative bias to the curve. The numerical data provided in Reference 6.10 is as follows:

Modulus of Elasticity, $E = 29{,}000{,}000$ psi
Initial strain hardening Modulus, $E_{st} = 900{,}000$ psi
Strain at which hardening begins, $\varepsilon_{st} = 0.014$ in/in

From Figure 1.1 in Reference 6.10, strain at minimum tensile strength is approximately 0.12 in/in. A stress-strain plot using these values is shown in Figure 5-1. The actual yield and ultimate stresses for the test specimens are higher than the values used in Figure 5-1.

Figure 5-1 - Stress-strain curve for A36 steel (Reference 6.10)

Strain [in/in]	Stress [psi]
0	0
0.00124(1)	36000
0.014	36000
0.030	50300
0.040	53070
0.080	57340
0.120	58000
0.200(2)	58000



Notes:
(1) This value is the nominal strain at yield, $\varepsilon_y = F_y/E$
(2) Value added for completeness

Reference 6.11 also established stress-strain data for typical structural steels for use in the development of the AISC LRFD Specification. The primary difference from Reference 6.11 is the use of a value for the initial strain hardening Modulus, E_{st} = 600,000 psi. This was based on 158 tests from a single laboratory for mix of A7, A36, and A441 steels. From Reference 6.10, properties for A441 are also provided, with an average value for the initial strain hardening Modulus, E_{st} = 700,000 psi. Reference 6.12 provides average stress-strain values for A7 steel in a similar manner to Reference 6.10. The properties used in Reference 6.12 are indicated to be from "many tests at Fritz Laboratory," at Lehigh University. Reference 6.12 gives the same strain value at which hardening begins, ε_{st} = 0.014 in/in as Reference 6.10 does for A36 steel. Reference 6.12 also gives a value for the initial strain hardening Modulus, E_{st} = 700,000 psi.

It is concluded that the average values in conjunction with the minimum specified yield strength and tensile strength from Reference 6.10 provide a technically acceptable stress-strain curve for A36 steel for use in essentially-elastic analysis of the trusses.

AISC N690 (which provides for allowable stress design only) allows increases in allowable stresses for load combinations including SSE, which in some cases (e.g. flexure about the major axis for compact sections, and flexure about the minor axis of doubly symmetric sections) provides an acceptance value which exceeds the minimum specified material yield strength. Stresses are determined on an elastic basis.

AISC N690 does provide higher ductility ratios for steel structures subjected to impact and impulse loads (e.g. missiles, pipe whip), which can be used in conjunction with seismic loads, however, their use is not directly permitted under AISC N690 for seismic load combinations in absence of the impact and impulse loads. Note, this is consistent with the guidance provided by the NRC in SRP 3. (Ref. 1.3), which previously provided ductility ratios for impact and impulse loads. For allowable ductility factors for steel, the current SRP 3.5.3 now just references AISC N690.

Although AISC N690 allows for the use of ductility factors only for loadings which include impact and impulse loads, the basis for the given ductility factors are from static tests on S and M sections (Ref. 6.13). The average maximum ductility factor attained was 26.4 when global buckling was precluded (Ref. 6.13). AISC N690 specifies an allowable ductility factor of 20 for closed sections and a reduced allowable ductility factor of 12.5 is specified for open cross-section flexural members (W, S, WT, etc.).

For a ductility factor of 20 with respect to the strain at yield (0.00124), an allowable strain of 0.0248, or approximately 2.5% is determined, and for a ductility factor of 12.5 relative to the strain at yield, an allowable strain of 0.0155, or approximately 1.5%, is computed. On the basis of the test results from (Ref. 6.13), an allowable strain limit 1.5% is used, which corresponds to the lower ductility factor of 12.5.

The 1.5% strain acceptance criterion is slightly larger than the strain value at which the strain-hardening begins (i.e. 1.4% in Figure 4-1). The use of the acceptance criteria is aligned with a model where truly elasto-plastic behavior is observed. The region where strain hardening occurs, i.e. strains between 1.4% and 12%, is considered as additional significant margin and ensure an acceptable performance of the trusses under extreme loading conditions.

5.6 Criteria for Containment Liner

As discussed in Section 3.2.2, the trusses may contact the liner at the first panel point. The concrete behind the liner provides bearing such that the liner is not subject to bending. Per ASME Code Section III, Division 1 Appendix F, Section F-1341.6 (Ref. 2.5), bearing stress does not need to be evaluated for loads for which Level D Service Limits are specified. However, to account for potential loss of concrete bearing due elevated load conditions, the liner has been evaluated for resulting localized stresses. The acceptance criteria for the containment liner are to ensure containment leak-tight integrity.

Typical nuclear design practice for a concrete containment liner is to apply strain limits. NUREG/CR-6906 (Ref. 1.18) summarizes experimental programs and analytical studies to investigate containment integrity. Tearing of steel liners under beyond design basis pressure and temperature was investigated. It was concluded that, in general, observed liner failures for steel-lined concrete containments resulted from local exceedance of the ductility limit due to strain concentration at anchors, penetrations and geometric discontinuities.

For the contact load condition the plastic analysis stress limits per ASME Code Section III, Division 1 Appendix F (Ref. 2.5) are applied to ensure leak-tight integrity. These stress limits are applied since the load occurs over a very small area (contact at the tip of the WT flange) compared to the overall liner plate area.

From Paragraph F-1341.2 of Ref. 2.5 (for plastic analysis), allowable limits are given in terms of stress as $0.9S_u$ for maximum primary stress intensity. Paragraph F-1322.3(b) of Ref. 2.5 indicates the allowable stress and strain limits given in Appendix F are based on an engineering stress-strain curve. It is noted that in Appendix F, all allowable limits are in terms of stress, with strains implied for a given stress level, relative to a valid material stress-strain curve.

To affirm that a safety margin is maintained, the truss contact load on the liner will not be higher than 2/3 of the maximum sustainable load per ASME Code Paragraph NB-3228.3.

To assess liner integrity under repeated seismic loading from a truss contact point, a finite element analysis of the liner for repeated loading/unloading cycles has been performed. The number of repeated loading/unloading cycles is determined from the shapes of the ground motion times histories used for the SSI analysis. At the beginning of each loading cycle, the deformed state of the liner at end of the previous cycle is used in order to determine the accumulation of the strains in the steel plate and the relative change in strain between each loading cycle. This relative change in strain between the first and last loading cycle is used as an additional tool to assess liner integrity. Postulated fatigue of the containment liner from the applied seismic loading is addressed in the supporting calculations, based upon the stress vs. cycles guidance provided in Figure I-9.1 of Ref. 2.5.

<u>Exceedance of permissible strain limits.</u>

For an analysis that accounts for the effect of the concrete in providing a restraint to the liner, exceedance of permissible strain is allowed as long as any corresponding reduction of the containment structure strength is acceptable.

For this analysis, the compressive strength of the concrete is based on the compressive strength from test data. ACI 318-63 (Ref. 2.2), Section 301, states:

> *"Specified compressive strength of concrete in pounds per square inch (psi). Compressive strength shall be determined by test of standard 6 x 12-in. cylinders made and tested in accordance with ASTM specifications at 28 days or such earlier age as concrete is to receive its full service load or maximum stress".*

The available test data [38] is based on 90-day strength tests and does not take credit for further age-hardening. While the 90-day strength test results provided in Ref. 5.3 represent the actual concrete compressive strength, they account for age-hardening beyond the specified 28-day strength. To limit the compressive strength from the 90-day tests, the compressive strength is limited to the 90th percentile of the actual test report data. This permits identifying actual limits, which supports evaluation of the containment liner. The extent of the localized concrete strain limit exceedance is verified by means of the maximum allowed compressive strain of 0.003 in/in (Ref. 2.2).

5.7 Seismic Fragility

To support the risk assessment of the trusses and attached/adjacent components, a high confidence of low probability of failure (HCLPF) capacity and seismic fragility parameters is determined.

The methodology and criteria to be applied for the HCLPF and fragility analysis are accepted industry practice for seismic probabilistic risk assessment (PRA) and are in compliance with the relevant guidance of Ref. 2.11, which is endorsed with clarifications by Regulatory Guide 1.200 (Ref. 1.6).

NUREG-0800 Section 19.2, "Review of Risk Information Used to Support Permanent Plant Specific Changes to the Licensing Basis: General Guidance", cites implementation of Regulatory Guide 1.200 as one way to ensure PRA technical adequacy. Therefore, the resulting HCLPF and fragility criteria are appropriate for support of a risk informed approach for a license amendment request (LAR).

<u>HCLPF Analysis</u>

The HCLPF capacity is determined using conservative deterministic failure margin (CDFM) criteria. The criteria are per Table A.1 of Ref. 6.7. The criteria are summarized below.

Load Combination:	Normal + Earthquake
Ground Response Spectrum:	Site GMRS
Seismic demand:	Perform seismic demand analysis as described in Table 4-2.
Damping:	Conservative estimate of median damping. Apply damping from ASCE 43-05 for appropriate stress level. For the trusses, 7% damping is applied as described in Table 4-2.
Structural Model:	Best estimate (median) + uncertainty variation in frequency
Soil-Structure Interaction	Best estimate (median) + parameter variation
Material Strength:	Code specified minimum strength or 95% exceedance of actual strength if test data are available
Static Capacity Equations:	Code ultimate strength (ACI), maximum strength (AISC), Service Level D (ASME), or functional limits. If test data are available to demonstrate excessive conservatism of code equation, then use 84% exceedance of test data for strength equation. For the trusses, AISC N690 is applied as described in Table 4-2.
Inelastic Energy Absorption:	For non-brittle failure modes and linear analysis, use appropriate inelastic energy absorption factor from ASCE 43-05 to account for ductility benefits, or perform nonlinear analysis and go to 95% exceedance ductility levels.
In-structure (Floor) Spectra Generation:	Use frequency shifting rather than peak broadening to account for uncertainty plus use conservative estimate of median damping.

The HCLPF earthquake load is calculated as follows:

$U = \text{Normal} + E_c$

where:
- U — ultimate strength or functional limit
- Normal — normal operating loads (dead load, live load expected to be present, etc.)

E_c HCLPF earthquake load, adjusted for factors such as inelastic energy absorption and load-redistribution (adjustment accounts for increasing the earthquake load determined from essentially elastic limits to a value that credits elastic-plastic transition)

The HCLPF earthquake loads are scaled from the seismic loads:

$$E_c = F_{ajd} \bullet SF_c \bullet E_{gm}$$

where: F_{adj} adjustment factors
SF_c component-specific scale factor that satisfies AISC N690
E_{gm} loads due to GMRS-based earthquake

The HCLPF is defined as the horizontal peak ground acceleration (PGA) corresponding to E_c. The GMRS-based earthquake has a PGA of 0.14g (Ref. 6.1). Therefore, the HCLPF is calculated as follows:

$PGA_c = F_{adj} \bullet SF_c \bullet (0.14g)$ component specific HCLPF, as peak ground acceleration

Fragility Parameters

The seismic fragility of a system, structure, or a component (SSC) is defined as the conditional probability of its failure at a given value of acceleration. The following parameters define the seismic fragility for any specific SSC:

PGA_{med} = median acceleration capacity
β_r = logarithmic standard deviation, randomness
β_u = logarithmic standard deviation, uncertainty
$\beta_c = SRSS(\beta_r, \beta_u)$ = logarithmic standard deviation, combined

The above parameters are tied to a lognormal probability distribution. PGA_{med} represents the best estimate of the seismic capacity (50% probability of failure). The β parameters address the variability of the estimate. Parameter β_r (randomness) accounts for sources of variability that cannot be reduced by more detailed studies or more data.

Parameter PGA_{med} corresponds to earthquake severity and is defined in terms of horizontal PGA at the GMRS control point. The fragility analysis accounts for propagation of earthquake ground motion to the SSC location and the resulting dynamic response of the item and its supporting structure.

Nominally the HCLPF is the capacity at which there is 95% confidence of less than 5% probability of failure. Fragility parameters are then produced by the Hybrid Method discussed in Section 6.4.1 of EPRI 1025287 (Ref. 6.5). The median capacity is estimated from the HCLPF value using the following equation (Ref. 6.7):

$PGA_{med} = PGA_c \bullet e^{2.3 \bullet \beta_c}$ (from Eq. 5-5 of Ref. 6.7)

PGA_c = HCLPF capacity, stated as peak ground acceleration

To produce the PGA_{med} for SSC, the HCLPF value is calculated and the corresponding βc value is estimated based on S-PRA experience. Per Table 6-2 of EPRI 1025287, $\beta_c = 0.35$ is applicable for structures and passive items mounted on ground or at low elevation within structures, $\beta_c = 0.45$ is applicable to active components mounted at high elevation in structures, and $\beta_c = 0.40$ is for all other SSCs. Since the trusses are passive structures mounted at a high in-structure elevation, it fits between the first two component types, and a $\beta_c = 0.40$ is applied. The randomness and uncertainty components of βc are set to 0.24 and 0.32 respectively per Table 6-2 of EPRI 1025287.

5.8 Thermal Analysis to Determine Probability of Failure vs Temperature

To support the risk assessment of the trusses with respect to the DBA, the following approach is utilized to establish the probability of failure vs temperature for the trusses.

The probability of failure is determined at the following temperature levels:

- The temperature at which liner contact first occurs.
- The temperature at which a fully plastic hinge first occurs in the bottom chord.
- The temperature that occurs as a result of a DBA.
- The temperature at which global truss failure is judged to occur.

The temperature at which global truss failure is judged to occur is determined following the approach outlined below, which is based on existing knowledge of truss behavior at various temperature levels.

- Determine the temperature at which a fully plastic hinge first occurs in the Detail 4 clip angle connection (Ref. 4.1), which is a critical connection.
- Limit the Detail 4 connection axial tension load to a value equal to the plastic capacity of the connection using nonlinear spring elements. The connection is considered to carry vertical and transverse loads until the radial displacement is significant. Once significant displacement is achieved, the connection is released.
- Increase the temperature until more Detail 4 clip angle connections reach their full plastic hinge capacities. Verify that the model is still stable (i.e., verify that the deflection at all connections that have been released are reasonable). Identify liner contact force. Verify that no other components in the truss have exceeded their capacity (i.e. the brace member or its connections adjacent to the largest liner force). If a different member or connection controls over the Detail 4 connection, release this member or connection in the next step.
- Repeat the above analyses until either an unacceptable liner force is reached or any element of the truss becomes unstable, as demonstrated in the supporting calculations.
- Using engineering judgment (as justified in the supporting calculations, Ref. 5.16 and 5.17), assign a probability of failure to the various temperatures. The temperature at which contact would initially occur is determined and assigned a very low probability of failure. Step 1 has the next to lowest probability and the temperature for truss instability/unacceptable liner contact force has the greatest probability. The existing design basis temperature (286°F) is assigned a probability of failure that takes into account the number of components at or near capacity, the deflections identified at the released connections, and any other properties of note.

6.0 Analyses in Support of License Amendment Request

6.1 Development of Site-Specific Horizontal GMRS

The site-specific horizontal GMRS was developed as discussed in Ref. 6.1 to support of the seismic screening to address the USNRC Post-Fukushima Near Term Task Force (NTTF) Recommendation 2.1. It was developed in accordance with RG 1.208 (Ref. 1.8) as allowed by SRP 3.7.1. Below is a summary of the methodology of Ref. 6.1 that were followed to determine the site-specific GMRS. The GMRS has been accepted by the NRC (Ref. 6.2).

6.1.1 Probabilistic Seismic Hazard Analysis

In accordance with the 50.54(f) letter (Ref. 6.4) and following the guidance in Ref. (6.5), a probabilistic seismic hazard analysis (PSHA) was completed using the recently developed Central and Eastern United States Seismic Source Characterization (CEUS-SSC) for Nuclear Facilities (Ref.1.15) together with the updated EPRI Ground-Motion Model (GMM) for the CEUS (Ref. 6.6). For the PSHA, a lower-bound moment magnitude of 5.0 was used, as specified in the 50.54(f) letter.

For the PSHA, the CEUS-SSC background seismic sources out to a distance of 400 miles (640 km) around Point Beach were included. This distance exceeds the 200 mile (320 km) recommendation contained in RG 1.208 and was chosen for completeness. Ref. 6.1 contains details on background sources included in the site analysis.

6.1.2 Base Rock Seismic Hazard Curves

Consistent with Ref. 6.5, base rock seismic hazard curves were not provided because the site amplification approach referred to as Method 3 (see subsection 6.1.11 for definition of Method 3) was used.

6.1.3 Site Response Evaluation

Following the guidance contained in Seismic Enclosure 1 of the 50.54(f) letter and Ref. 6.5 for nuclear power plant sites that are not sited on hard rock, which is defined as rock with a shear wave velocity of 2.83 km/sec (1.76 miles/sec), a site response analysis was performed for Point Beach.

6.1.4 GMRS Control Point

Per Ref. 6.5 guidance, the SSE Control Point was taken to be at the elevation of the highest foundation of key structures, which is elevation +8ft (+2.4 m).

6.1.5 Shear Modulus and Damping

No site-specific nonlinear dynamic soil material properties were available for Point Beach. The soil material over the upper 83ft (25 m) was assumed to have behavior that could be modeled with either EPRI cohesionless soil or Peninsular Range G/Gmax and hysteretic damping curves (Ref. 6.5). Consistent with Ref. 6.5, the EPRI soil curves (model M1) were considered to be appropriate to represent the more nonlinear response likely to occur in the materials at the site. The Peninsular Range (PR) curves (Ref. 6.5) for soils (model M2) was assumed to represent an equally plausible alternative more linear response across loading levels.

6.1.6 Kappa

Base-case kappa estimates were determined using Section B-5.1 .3.1 of Ref. 6.5 for sites with less than 3,000 ft. (1,000m) of soil. For soil sites with depths less than 3,000 ft. (1,000m) to hard rock, a mean base-case kappa may be estimated based on total soil thickness of 83ft (25m) with the addition of the hard basement rock value of 0.006s (Ref. 6.5). For base-case profiles P1, P2, and P3 the kappa contributions from the profiles was 0.002s, 0.003s, and 0.001s respectively. The total kappa values, after adding the hard reference rock value of 0.006s, were 0.008s, 0.009s, and 0.007s respectively. Epistemic uncertainty in profile damping (kappa) was considered to be accommodated at design loading levels by the range of damping (kappa) provided by the multiple (2) sets of G/Gmax and hysteretic damping curves.

6.1.7 Randomization of Base Case Profiles

To account for the aleatory variability in material properties that is expected to occur across a site at the scale of a typical nuclear facility, variability in the assumed shear-wave velocity profiles was incorporated in the site response calculations. For Point Beach, random shear wave velocity profiles were developed from the base case profiles. Consistent with the discussion in Appendix B of Ref. 6.5, the velocity randomization procedure made use of random field models which describe the statistical correlation between layering and shear wave velocity. The default randomization parameters developed in Ref. 6.8 for USGS A site conditions were used for the site. Thirty random velocity profiles were generated for each base case profile. These random velocity profiles were generated using a natural log standard deviation of 0.25 over the upper 50ft and 0.15 below that depth. As specified in Ref. 6.5, correlation of shear wave velocity between layers was modeled using the USGS A correlation model. In the correlation model, a limit of ±2 standard deviations about the median value in each layer was assumed for the limits on random velocity fluctuations.

6.1.8 Input Spectra

Consistent with the guidance in Appendix B of Ref. 6.5, input Fourier amplitude spectra were defined for a single representative earthquake magnitude (M 6.5) using two different assumptions regarding the shape of the seismic source spectrum (single-corner and double-corner). A range of 11 different input amplitudes (median peak ground accelerations (PGA) ranging from 0.01 to 1.5 g) were used in the site response analyses. The characteristics of the seismic source and upper crustal attenuation properties assumed for the analysis of the Point Beach site were the same as those identified in Tables B-4, B-5, B-6, and B-7 of Ref. 6.5 as appropriate for typical CEUS sites.

6.1.9 Methodology

To perform the site response analyses for the Point Beach site, a random vibration theory (RVT) approach was employed. This process utilized a simple, efficient approach for computing site-specific amplification functions and is consistent with existing NRC guidance and Ref. 6.5. The guidance contained in Appendix B of Ref. 6.5 on incorporating epistemic uncertainty in shear-wave velocities, kappa, non-linear dynamic properties and source spectra for plants with limited at-site information was followed for the site.

6.1.10 Amplification Functions

The results of the site response analysis consist of amplification factors (5% damped pseudo absolute response spectra) which describe the amplification (or de-amplification) of hard reference rock motion as a function of frequency and input reference rock amplitude. The amplification factors were represented in terms of a median amplification value and an associated standard deviation (sigma) for each oscillator frequency and input rock amplitude. Consistent with Ref. 6.5, a minimum median amplification value of 0.5 was employed in the analysis.

6.1.11 Control Point Seismic Hazard Curves

The procedure to develop probabilistic site-specific Control Point hazard curves used in the analysis followed the methodology described in Section B-6.0 of Ref. 6.5. This procedure (referred to as Method

3) computed a site-specific Control Point hazard curve for a broad range of spectral accelerations given the site-specific bedrock hazard curve and site-specific estimates of soil or soft-rock response and associated uncertainties. This process was repeated for each of the seven specified oscillator frequencies. The dynamic response of the materials below the Control Point was represented by the frequency and amplitude-dependent amplification functions (median values and standard deviations) developed and described in the previous section.

6.1.12 Control Point Response Spectra

The Control Point hazard curves described above were used to develop uniform hazard response spectra (UHRS) and the GMRS. The UHRS were obtained through linear interpolation in log-log space to estimate the spectral acceleration at each oscillator frequency for the 1E-4 and 1E-5 per year hazard levels.

6.2 Development of In-Structure Seismic Response at Dome Truss Supports

A SSI analysis was performed to generate acceleration time histories at containment elevation 125 ft. for use in analyzing the trusses, containment spray lines, and attached components. Below is a flowchart (as shown in Ref. 5.13) that shows the methodology for the development of in-structure seismic response at containment structure elevation 125 ft. (truss support actual elevation is 124'-5¼" per drawing Ref. 4.1). The steps to develop the in-structure seismic response are discussed in the sections to follow.

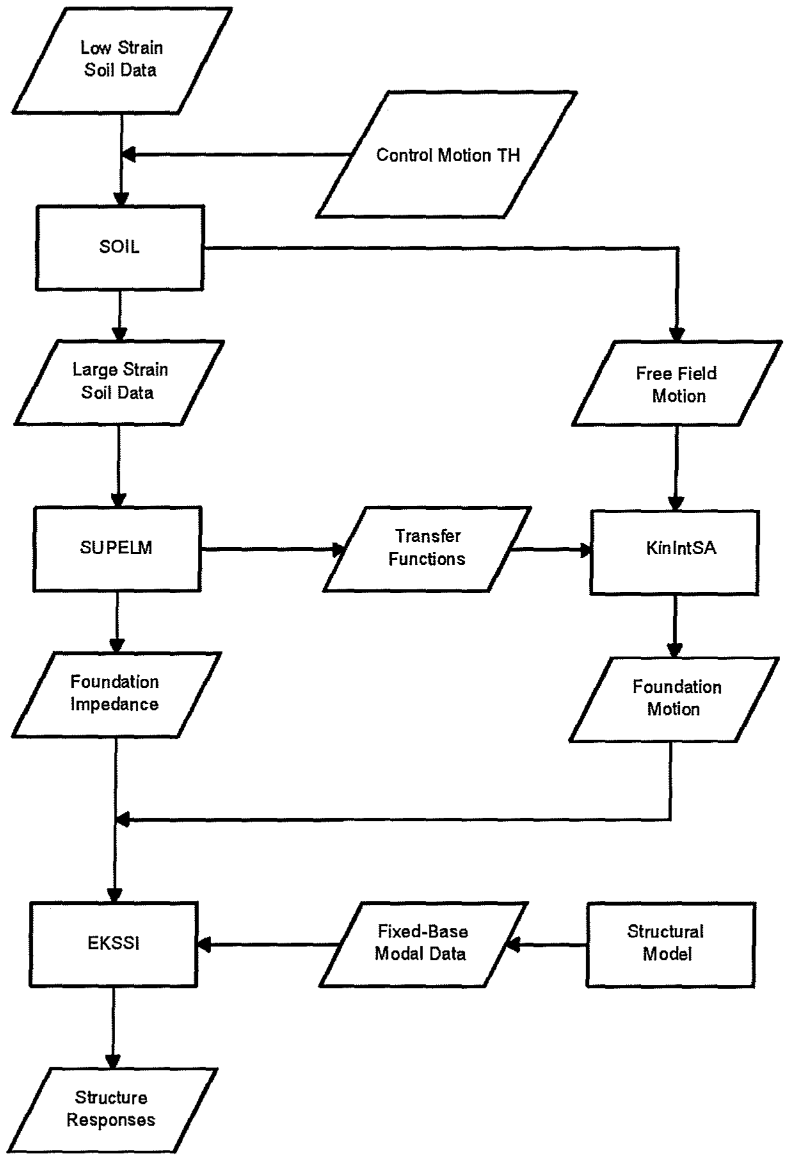


Figure 6-1 – Flowchart of SSI Analysis Methodology

The development of the ground motion time histories is documented in Ref. 5.12. The SSI analysis is documented in Ref. 5.13.

6.2.1 Development of Ground Motion Time Histories

The development of the ground motion time histories (Ref. 5.12) for the SSI analysis (Ref. 5.13) met Section 2.4 of ASCE/SEI 43-05 with the limitations identified in NUREG/CR-6926. This is consistent with NUREG-0800, Standard Review Plan (SRP) Section 3.7.1 (Ref. 1.1), Acceptance Criterion 1B, Option 2, which requires that (a) through (d) of Option 1, Approach 2, are met (a comparison of requirements between Section 2.4 and SRP 3.71 is provided in Ref. 6.9).

The vertical GMRS was developed based on the horizontal GMRS using mean V/H ratios for A-Soft sites per Ref. 6.3. The use of the A-soft site class V/H ratios is based on Vs30 values close to 820 ft./sec per Ref. 6.1 and GMRS peak ground acceleration less than or equal to 0.2g.

The method for TH generation was as enumerated below (Ref. 5.12).

1. Five sets of two horizontal and one vertical time histories (TH) were developed from seed motions which were obtained from NUREG/CR-6728 (Ref. 1.16). Seed motions were selected from multiple Magnitude/Distance (M/D) bins deemed appropriate for the site. The selected seed motions met the following parameters:

    a. Time step no greater than 0.005 seconds to ensure a Nyquist frequency of at least 100Hz. Meets ASCE 43-05, Section 2.4(a).
    b. Strong motion duration (taken as the time for Arias Intensity growth from 5% and 75%) is longer than 6 seconds (shown to range from 8.12 seconds to 18.26 seconds per Ref. 5.12).
    c. Total duration no shorter than 20 seconds. Meets ASCE 43-05, Section 2.4(a).
    d. Spectral shape of seed motion similar to the target GMRS spectral shapes. The GMRS spectral shape is primarily defined by a broad range of amplification with a peak at around 12.5 Hz. Spectral shapes were considered similar to the GMRS if the amplified range was reasonably broad and occurred at frequencies between about 5 and 20 Hz.

2. Seed motion records were modified over a series of iterations to approximate the target spectra. For each iteration, the time history is converted to the frequency domain and Fourier amplitudes are adjusted based on the difference between the time history response spectrum and the target spectrum. Throughout the iteration process, the phase angle is kept unchanged (Meets ASCE 43-05, Section 2.4).

    After each iteration, 5% damped response spectra (RS) were developed 100 points per frequency decade (Meets ASCE 43-05, Section 2.4(b)) and the average of all response spectra for a given direction was compared to the target spectrum over a frequency range of interest of 0.3 Hz to 40 Hz. While the frequency range of interest is limited to 0.3 Hz to 40 Hz, all response spectra were developed for the frequency range of 0.1 Hz to 100 Hz for consistency with the frequency range of the input GMRS. The following enveloping criteria from ASCE/SEI 43-05 were considered:

Criterion	Outcome
No frequency point shall fall more than 10% below the target spectrum Meets ASCE 43-05, Section 2.4(c)	The minimum ratio of TH RS to target RS was 0.94
No frequency point shall exceed the target spectrum by more than 30% Meets ASCE 43-05, Section 2.4(d)	The maximum ratio of TH RS to target RS was 1.07
No more than 9 adjacent frequency points shall fall below the target spectrum Meets ASCE 43-05, Section 2.4(c)	The maximum number of consecutive points that fell below the target RS was 7

3. The statistical independence of the THs were checked to verify that the absolute value of correlation coefficient did not exceed 0.16 for any pair. Whereas Section 2.4(f) of ASCE/SEI 43-05 stipulated that the correlation coefficient should not exceed 0.3, NUREG/CR-6926 stipulated that a minimum value of 0.16 should be followed, which is consistent with SRP 3.7.2.

   The maximum correlation coefficient for TH pairs was 0.138, which met the 0.16 limit.

6.2.2 Seismic Analysis of Free-Field for Large Strain Soil Properties

Computer program SOIL, which is a module of ACS-SASSI (Ref. 7.1), was used (Ref. 5.13) to determine the time histories at the free-field surface from time history inputs at an elevation of +8'. It performs non-linear site response analysis under vertically propagating S waves using an equivalent-linear iterative model for soil hysteretic non-linear behavior. It computes the dynamic response of viscoelastic, horizontally layered soils over elastic or rigid rock to seismic ground motions. It computes the soil strains in the layers under the defined free field motion using best estimate (BE) small strain profile of the layered soil. Based on the computed soil strain, the changes to the shear wave velocity along with Poisson's ratio and the effective damping ratio are computed.

For each time history set, the large-strain best estimate soil properties are requested as output. It uses degradation curves (Ref. 6.14) to carry out iterations on soil properties to account for large strain effects on shear modulus and damping. A degradation curve defines the variation of shear modulus or damping with shear strain. In addition, SOIL is run with the large-strain properties in order to convolve the time histories to the free-field surface for use as input to later modules.

In addition to the best estimate soil properties, lower bound (LB) and upper bound (UB) large strain soil properties were calculated. The LB and UB shear modulus values were taken as 1/2 and 2 times the initial BE shear modulus values, respectively, which is taken as a maximum uncertainty value for well-investigated sites based on SRP 3.7.2.

All soil damping ratios were less than 0.15 and UB large-strain shear moduli were limited to not be less than the BE low-strain shear moduli, as required by SRP 3.7.2.

6.2.3 Soil Impedance and Kinematic Interaction of the Foundation

Computer program SUPELM (Ref. 7.2) was used (Ref. 5.13) to determine impedance functions for the underlying soil based on a cylindrical foundation. Impedance values were determined for frequencies ranging from 0.0 to 80 Hz in steps of 0.1 Hz for Horizontal Translation, Rocking, Coupled Horizontal Translation and Rocking, and Vertical Translation. For vertical impedance, a damping value equal to half of the determined horizontal value is applied as a reasonable approximation between low- and large-strain shear damping, both of which are commonly used for compression wave damping. The vertical and rotational stiffness of the piles was calculated and applied in the soil impedance model. Impedances were combined into a 5x5 matrix that accounts for all degrees of freedom that would be considered for the SSI model (note: due to the cylindrical shape of the foundation, no rotational response about the vertical direction need be accounted for).

SUPELM was also used to compute transfer functions to account for kinematic interaction. These transfer functions were then used to deconvolve the free-field time histories and determine the time histories for the motion at the foundation base.

The analyses described above were performed for each of the BE, LB, and UB cases (see Ref. 5.13 for further detail).

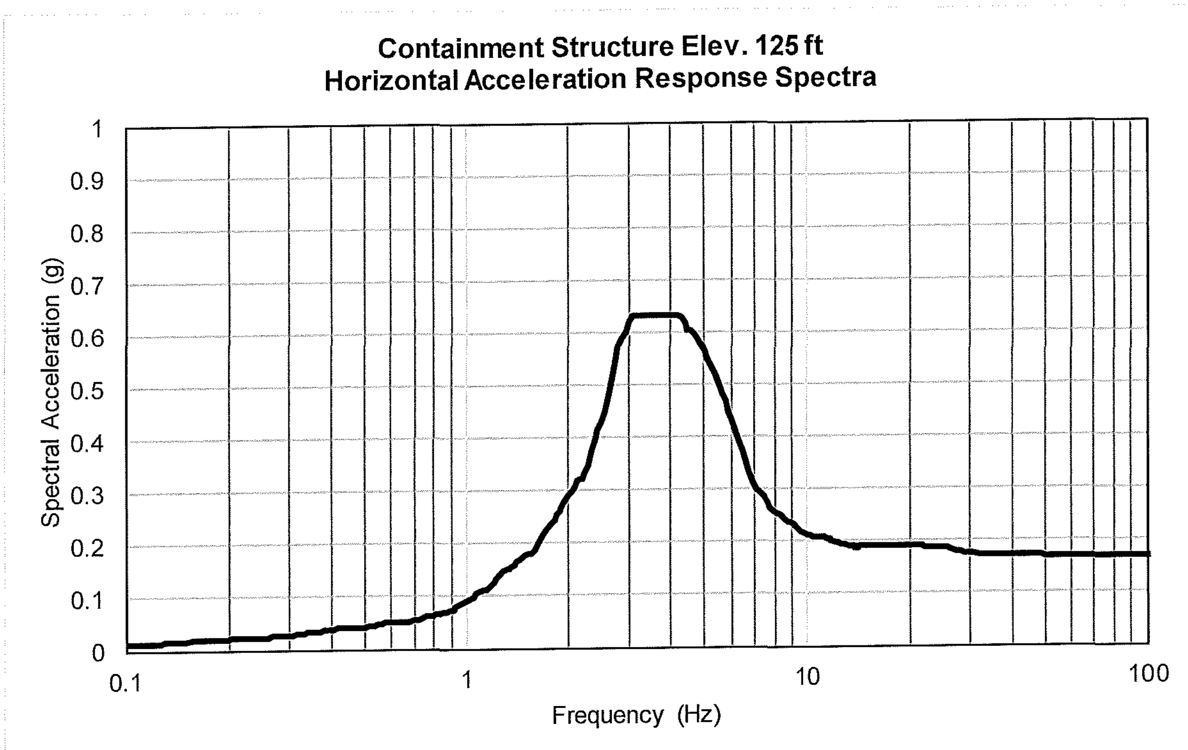
6.2.4 Soil-Structure Interaction Analysis

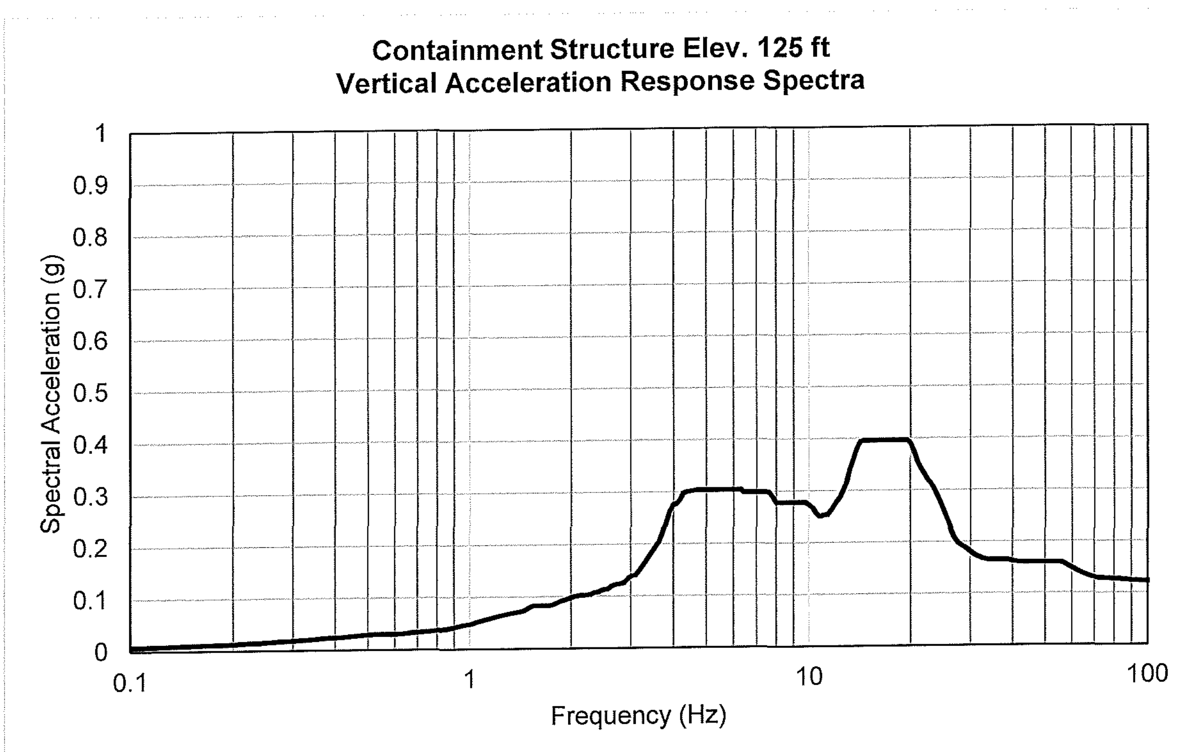
Computer program EKSSI (Ref. 7.3) was used to combine the soil and structure models. The structural model, including the magnitudes and locations of nodal masses, was that per Ref. 5.1. The structural model was validated to confirm that it computed the same dynamic results as those of Ref. 5.1. The damping used for the prestressed concrete containment structure was 5% (see Section 5.3). The inputs were the frequency-dependent soil impedance matrices and the time history seismic motions at the foundation base. The outputs were the time histories at containment structure elevation 125 ft. for each of the BE, LB, and UB cases for all (five) of the input time histories in each of the two horizontal and vertical directions.

6.2.5 In-Structure Seismic Response at Dome Truss Supports

Response spectra (at 7%, see Section 5.3) were developed for the time histories developed as discussed in Section 6.2.4 for each of the BE, LB, and UB cases. The five response spectra for each of the BE, LB, and UB cases were averaged for each direction. The two horizontal response spectra for the BE, LB, and UB cases were enveloped and broadened by 15% in accordance with RG 1.122 (Ref. 1.9) to develop one horizontal input spectra. The vertical response spectra for the BE, LB, and UB soil conditions were enveloped and broadened by 15% in accordance with RG 1.122 to develop one vertical input spectra. Per Appendix A.5, "Seismic Design Analysis", of the UFSAR (Ref. 3.1) the seismic loads are the maximum loads due to the horizontal earthquake combined with the vertical loads due to the earthquake.

The final in-structure seismic response spectra are shown on the following page.


Containment Structure Elev. 125 ft
Horizontal Acceleration Response Spectra
Spectral Acceleration (g)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.1
1
10
100
Frequency (Hz)


Containment Structure Elev. 125 ft
Vertical Acceleration Response Spectra
Spectral Acceleration (g)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.1
1
10
100
Frequency (Hz)

6.3 Liner Analysis

The liner analysis is documented in Ref. 5.7. In addition to determining the liner capacity in accordance with the criteria discussed in Section 5.6, Ref. 5.7 developed the stiffness of the liner contact points and provided a load vs. deflection curve that was used in the truss analyses.

A finite element model (FEM) was developed to capture the local behavior of the liner at the contact point and the local behavior of the concrete behind the liner. The steel liner was modeled as a nonlinear material with strain-hardening behavior. The concrete in the containment shell was modeled as an elasto-plastic material because of the confinement provided by (i) the significant amount of concrete surrounding the region where contact is considered and (ii) by the steel liner itself.

The FEM consisted of a containment shell cylindrical section with 8" diameter in plan view (i.e. looking towards the liner) by 2' deep, and was considered sufficient to capture the local behavior at the contact point (validated by negligible strains present at the model boundaries). The actual model analyzed is a quarter model to take advantage of the double symmetry of the full model. The initial liner contact area loaded by the truss is approximated by a square shape (19/32" x 19/32"). The contact area dimensions and truss shape used in the analysis were based on the truss top chord geometry at the contact point.

The concrete directly behind the contact point is expected to have strains that exceed permissible limits due to the highly localized stresses in the relatively small contact area. However, the localized area exceeding permissible strain is confined by the unstressed/undamaged surrounding concrete (90" deep at that location) and the liner. Although the concrete exhibits strains beyond permissible limits, it still has the capacity to carry the applied load.

The containment shell/wall concrete compressive strength (f'c) is defined as 7.2 ksi based on a statistical analysis of available Unit 2 90-day strength data (Ref. 5.3), since not all individual Unit 1 strength data was included in Ref. 5.3. The concrete compressive strength of 7.2ksi corresponds to the value with the 90% probability of being exceeded for the standard normal distribution. As discussed in Ref. 5.7, the maximum increase between 28-day and 90-day compressive strength is approximately 10%. To account for the 28-day strength, the 90-day compressive strength is reduced by 10%, and the resulting compressive strength is shown to be less than 7.2 ksi. The results for the liner analysis using the compressive strength as obtained from the Unit 2 data, was used for both Units 1 and 2, and justified by showing that the mean compressive strengths for the Units 1 and 2 strength data were similar.

The finite element analysis (FEA) is performed by applying a force controlled load. Two loading conditions were analyzed:

- Ten loading/unloading cycles for the repeated contact load due to seismic interaction from the dome truss. A load of 52.8kips is applied 10 times in 10 seconds. The magnitude of the load is determined in an iterative, i.e., trial and error, process based on the acceptance criteria of stress intensity not to exceed 0.9Su. The allowable load is defined as the maximum load that could be applied without exceeding the acceptance criteria at any of the peaks of the 10 loading/unloading cycles.

  The number of peaks used for the repeated loading are based on strong motion duration and total duration as described in calculation Ref. 5.12. The time histories selected as seed ground motions in Ref. 5.12 have a strong motion duration longer than 6 seconds and a total duration no shorter than 20 seconds. A conservative bounding case is considered by defining 10 loading/unloading cycles in 10 seconds which is half of the total duration and almost twice as the strong motion duration defined as minimum values for the seed motions in Ref. 5.9.

- A single loading cycle ramped up to 120kips load in ten seconds.

The results of the FEA showed that the maximum liner contact is 52.8 kips (which was also applied as the peak of the repeated load). To verify that the integrity of the steel liner would not be compromised in the 10 repeated cyclic loading case, the stress corresponding to the maximum strain range was determined. The allowable number of cycles corresponding to the stress was determined to be 7000 from the fatigue curve (Ref. 2.5, Figure I-9.1), and compared to the applied number of cycles. The usage factor was 0.00143, which is negligible.

In addition to the fatigue usage factor, the relative increase/change in strains due to the repeated contact load were computed. The maximum relative variation/increase of the strains between the first and last loading cycles was 1.68% for the first principal strain. This slight variation in the strains reassures that the liner integrity is not compromised for the maximum allowable load of 52.8 kips.

For the single loading cycle ramped up to 120 kips, the slope of the load vs deflection curve from 20 kips to 120 kips does not change direction, i.e., the curve does not plateau at 120 kips. Therefore, the maximum sustainable load is larger than 120kips. This is mainly explained by the fact that the contact area grows as the corner joint in the dome truss displaces into the liner/wall assembly. Also, as the dome truss displaces, the extent of the localized damage in the concrete grows and allows for the additional load to be spread out in a larger area. The fact that the steel is a material approximately 4 times stronger and 6 times stiffer than the concrete also plays an important role in the growing contact area as the truss moves into the liner/wall assembly.

The allowable contact load of 52.8 kips based on a maximum stress intensity is less than two-thirds (80 kips) of the maximum applied load of 120 kips, as noted in Paragraph NB-3228.3 of Ref. 2.5.

The results of the liner analysis verify that the liner maintains its structural integrity under repeated contact loading from the trusses for a load not to exceed 52.8 kips.

6.4 Truss Probability of Failure vs Temperature and Seismic Fragility Analysis

Analyses to determine the probabilities of failure vs temperature and seismic fragilities for each dome truss were performed. The acceptance criteria discussed in Section 4.0 were used to support the analyses. The data are used to support the PRA analysis as part of the risk-informed LAR.

6.4.1 Probability of Failure vs Temperature

6.4.1.1 Unit 1

The analyses to determine the probability of failure vs temperature for the Unit 1 truss are documented in Ref. 5.16. The existing truss was modeled with the supports released radially, and nonlinear springs modeled at the first panel points. The force vs. deflection curve for the nonlinear springs was that developed in Ref. 5.7 as discussed in Section 6.3. The nonlinear spring for each panel point was adjusted to account for the existing gap per Ref. 3.11.

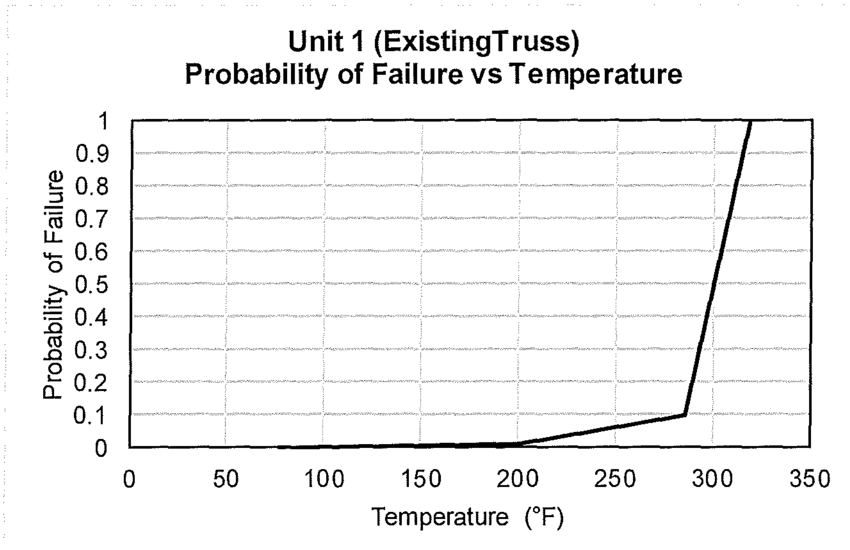
The following procedure was used to develop the probability of failure vs temperature curve:
1) Determine the temperature at which liner contact first occurs.
2) Determine the temperature at which a fully plastic hinge first occurs.
3) Assess the dome truss at the DBA thermal loading (286°F).
4) Increase the temperature, including the effects of plastic behavior in additional dome truss components, until instability is observed in the dome truss.

In each of Steps 2 through 4, dome truss components were checked against the acceptance criteria of N690 to determine the component that was the most highly loaded. Components that did not meet the acceptance criteria of N690 were checked against their ultimate capacities. If the load in a truss component exceeded its ultimate capacity, it was removed from the computer model for the next analysis and the truss was assessed for stability. In each analysis, the liner contact force was compared to that developed in Ref. 5.7 as discussed in Section 6.3.

A probability of failure was assigned to each of the temperature levels determined for Steps 1 – 4 above. The assignment of probability of failure of the dome truss was based on the increased likelihood of failure at the maximum stress level. The probability of failure vs temperature for the Unit 1 dome truss is as follows:

Unit 1 (Existing Truss) Probability of Failure vs Temperature		
Description	Temperature (°F)	Probability of Failure
First Contact with Liner	78	1E-12
First Fully Plastic Hinge	201	0.01
Design Basis Temperature	286	0.10
Capacity Limit	318	0.99

The plot of the probability of failure vs temperature curve is shown on the following page.



The thermal analyses showed that at DBA thermal levels, the contact force with the liner exceeded the acceptance criteria discussed in Section 4.0, and several Detail 4 (Ref. 4.1) clip angles were subject to plastic behavior. As discussed in Section 3.3.2, evaluations for operability demonstrated that the truss would maintain structural integrity under these conditions.

To bring liner contact forces and component stresses to acceptable levels, the Unit 1 truss requires modification. The high loads/stresses are due to the contact with the liner that prevents the dome truss from expanding unimpeded under thermal loads. Reducing the liner contact loads and stresses in truss components would be achieved by trimming the first panel point at a sufficient number of locations to reduce the thermal loads in the truss such that the acceptance criteria as discussed in Section 4.0 are met. The analysis of the modified Unit 1 truss is discussed in Section 6.5.1.

As discussed in Ref. 5.16, the probability of failure vs temperature curve of the modified Unit 1 truss is bounded by that of the existing Unit 2 truss (see Section 6.4.1.2 below).

6.4.1.2 Unit 2

The analyses to determine the probability of failure vs temperature for the Unit 2 truss are documented in Ref. 5.17. The existing truss was modeled with the supports released radially, and nonlinear springs modeled at the first panel points. The force vs. deflection curve for the nonlinear springs was that developed in Ref. 5.7 as discussed in Section 6.3. The nonlinear spring for each panel point was adjusted to account for the existing gap per Ref. 3.12.

The following procedure was used to develop the probability of failure vs temperature curve:

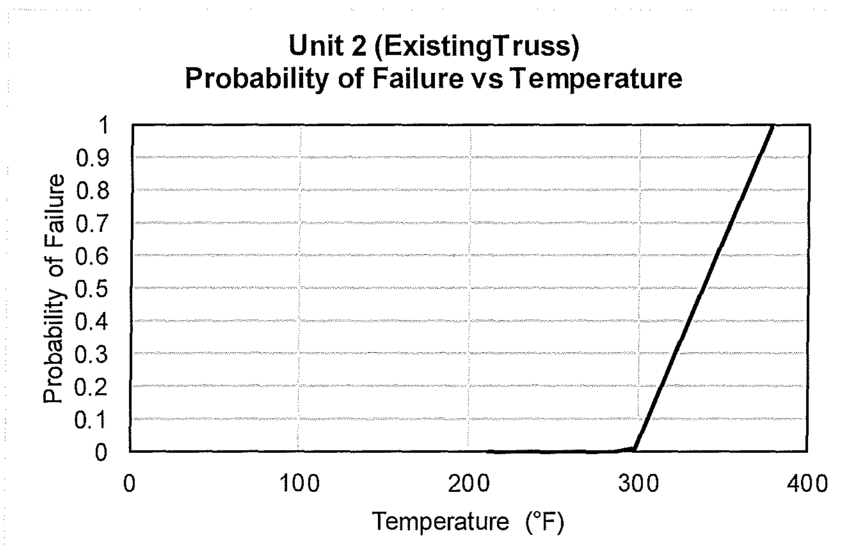
1) Determine the temperature at which liner contact first occurs.
2) Assess the dome truss at the DBA thermal loading (286°F).
3) Determine the temperature at which a fully plastic hinge first occurs.
4) Increase the temperature, including the effects of plastic behavior in additional dome truss components, until instability is observed in the dome truss.

In each of Steps 2 through 4, truss components were checked against the acceptance criteria of N690 to determine the component that were the most highly loaded. Components that did not meet the acceptance criteria of N690 were checked against their ultimate capacities. If the load in a truss component is exceeded its ultimate capacity, it was removed from the computer model for the next analysis and the dome truss was assessed for stability. In each analysis, the liner contact force was compared to that developed in Ref. 5.7 as discussed in Section 6.3.

A probability of failure was assigned to each of the temperature levels determined for Steps 1 – 4 above. The probability of failure of the dome truss was based on the increased likelihood of failure at the maximum stress level. The probability of failure vs temperature for the Unit 2 truss is as follows:

Unit 2 (Existing Truss) Probability of Failure vs Temperature		
Description	Temperature (°F)	Probability of Failure
First Contact with Liner	211	1.0E-12
Design Basis Temperature	286	0.001
1st Fully Plastic Hinge	298	0.01
Capacity Limit	378	0.99

The plot of the probability of failure vs temperature curve is shown below.



No structural challenges were identified for the existing Unit 2 truss at DBA temperature levels.

6.4.2 Seismic Fragility

The calculation of the seismic fragilities is documented in Ref. 5.14, and shows that the seismic fragilities of the trusses is lower than the seismic fragilities of the attached/adjacent components, and therefore, controls the PRA analysis. To support the PRA analysis, the seismic fragility of the trusses, Unit 1 with limited modifications and Unit 2 unmodified, were calculated. Additionally, the seismic fragility of trusses modified for full code compliance was calculated. The seismic fragilities of the trusses are summarized below.

6.4.2.1 Unit 1 with Limited Modifications and Unit 2 Unmodified

The dome trusses were analyzed (Ref. 5.10 & 5.11) to determine the seismic demand at which the stress in a truss component is equal to the acceptance criteria of AISC N690. The Unit 1 truss was analyzed in the configuration with limited modifications (six first panel point locations require trimming) as discussed in Section 6.5.1. A scale factor of 0.44 was calculated to apply to the GMRS PGA of 0.14g such that the equivalent PGA result in a maximum stress that satisfies AISC N690. The maximum stress occurs in the T2 truss bottom chords.

To calculate the HCLPF, the following additional factors were considered the most relevant, using the guidance from Ref. 6.7 and Ref. 6.15, and applied to the equivalent PGA:

<u>Frequency Uncertainty</u>

A ±10% frequency uncertainty for the truss was considered applicable following the guidance of Ref. 6.15. Based on review of applied BE, LB and UB 7% damped spectra of Ref. 5.13, a ±10% shift in the truss frequency would not significantly increase the spectral acceleration at the truss lateral frequency. Furthermore, the scale factors from Ref. 5.10 & 5.11 were obtained from a response spectra analysis that used response spectra that was broadened by ±15%. Therefore, no additional adjustments to frequency uncertainty are required.

Ffr = 1.0

<u>Load Redistribution</u>

The truss supports allow free motion in the radial direction but restrain motion in the tangential direction. This design accommodates thermal contraction and expansion but results in a flexible structure because the load path to the tangential reactions is through the inter-truss bracing members.

The trusses that are positioned perpendicular to the load direction would reach yield prior to other trusses. Also, the T2 truss members reach yield before the T1 truss members. Additional load can be carried by lower-stressed trusses. However, the capability for redistribution is limited by acceptable rotation of plastic hinges (see ASCE 43-05 Table 5-3). A factor of 1.2 is judged reasonable for a HCLPF, based primarily on load transfer to closely adjacent trusses prior to the final acceptable limit state. Load transfer to trusses beyond the adjacent trusses has limited benefit since the tangential reaction load is skewed from the load direction.

Frd = 1.2

<u>Inelastic Energy Absorption</u>

The failure mode of the trusses can be considered ductile and an inelastic energy absorption factor of 1.25 or greater is justified based on the review of the factors listed in ASCE 43-05 Table 8-1 for "Equipment Supports". The Limit State B factor is judged reasonable since this is associated with "Moderate Permanent Distortion" and this is estimated to be achievable for the truss. Therefore, apply:

$F\mu = 1.5$

Using the above methodology and the lower scale factor, the following HCLPF is calculated:

$PGA_c = F_{fr} \cdot F_{rd} \cdot F\mu \cdot (0.44) \cdot (0.14g) = 0.11g < \text{GMRS PGA} = 0.14g$

An equivalent static analysis was performed in Ref. 5.10 & 5.11 using spectral accelerations based on the GMRS PGA of 0.14g. In addition to comparing results to the strength limits of Ref. 2.7, for members where stresses exceed code strength limits, strain acceptance criteria limit of 1.5% were applied as discussed in Ref. 6.9. The purpose of these analyses was to show that the trusses maintained structural integrity for a GMRS PGA of 0.14g.

The analyses documented in References Ref. 5.10 & 5.11 show that the truss components meet the acceptance criteria in Ref. 2.7 for the seismic demand of Ref. 5.13. Considering that the acceptable limits are met with margin (see Sec. 6.5.2 below) a HCLPF can be determined based on the above results by applying the following factors:

$SF = 1.0$

$Ffr = 1.0$

$Frd = 1.0$

$F\mu = 1.2$ (since Ref. 5.10 & 5.11 already account for some ductility, a reduced inelastic energy absorption factor is applied compared to the factor used for the elastic capacity)

Using the above methodology and the lower adjusted factors, the following HCLPF is calculated:

$PGA_c = F_{fr} \cdot F_{rd} \cdot F\mu \cdot (SF) \cdot (0.14g) = 0.168g$

Based on the above results, the PGAc calculated from the equivalent static analysis is higher.

For a PGA of 0.168g, the seismic fragility is:

$PGA_{med} = (0.168g) * e^{2.3 * 0.40} = 0.42g$

6.4.2.2 Modifications to Meet AISC N690

To support the assessment of the ΔCDF as discussed in Ref. 1.10, the HCLPF of modified trusses was determined. The modifications were those that would increase the strength capacity of components to meet the acceptance criteria of AISC N690. The modifications comprise of the following:

- Installation of inward radial restraints at the six T1 truss supports. These restraints provide support in one direction (inward), yet allows for radial thermal growth.
- Modifications to the bottom chords to increase the strength of these members.
- Trimming the first panel point at 14 locations for Unit 1 and 11 locations for Unit 2 such that no contact occurs with the liner under either seismic or DBA thermal event.

The truss analyses are documented in Ref. 5.24. The Unit 2 truss was analyzed since, as documented in Ref. 5.24, it bounds the Unit 1 truss analysis. Seismic input to the analysis was the in-structure seismic demand discussed in Section 6.2.5. The analysis concluded that the PGA at which the trusses reach their strength capacities is 0.212g. The strength capacity is controlled by the shear yield capacity of the bolts that connect the truss support beam to the posts that are embedded in the containment concrete wall (see Detail 2 of Ref. 4.1).

Since no further load redistribution or inelastic energy absorption can be attained if the trusses were to be subjected to a PGA higher than 0.212g, the HCLPF is considered equal to 0.212g, and the seismic fragility is:

$$PGA_{med.modified} = (0.212g) * e^{2.3 * 0.40} = 0.53g$$

6.5 Analysis of Trusses and Attached Components for Structural Integrity

6.5.1 Truss Thermal Analysis

As discussed in Section 6.4.1.1 the existing Unit 1 truss requires modification to reduce the liner contact loads and stresses in truss components Reducing the liner contact loads and stresses in truss components would be achieved by trimming the first panel point at sufficient locations such that the acceptance criteria discussed in Section 4.0 are met.

Ref. 5.8 documents the thermal analysis of the Unit 1 dome truss and concludes that six first panel point locations require trimming such that the acceptance criteria discussed in Section 4.0 are met.

As discussed in Section 6.4.1.2, no structural challenges were identified for the existing Unit 2 truss at DBA temperature levels, therefore, no modifications are required. Ref. 5.9 documents the thermal analysis of the existing Unit 2 truss.

6.5.1.1 Analysis Methodology

Each dome truss was modeled with the supports released radially, but restrained laterally by a spring that accounts for the combined effect of the lateral and rotational stiffness of the support beams. Nonlinear springs were modeled at the first panel points. The force vs. deflection curve for the nonlinear springs was that developed in Ref. 5.7 as discussed in Section 6.3. The nonlinear spring for each panel point was adjusted to account for the existing gap per Ref. 3.11 & 3.12.

Boundary conditions specific to the Unit 1 dome truss:

- Nonlinear springs were not modeled at the trimmed first panel points because they were trimmed sufficiently such that no contact with the liner occurs at the DBA temperature.
- Since the thermal analysis showed that the second panel point at the truss identified as number 3 in Ref. 3.11 would contact the liner at the DBA temperature level, a nonlinear spring was modeled at this location.

The dome trusses were analyzed for dead load and thermal loads due to a temperature change from 70°F to 286°F. Dead load and thermal loads from Ref. 5.25 and 5.26 for the containment spray pipes were included in the analyses.

6.5.1.2 Analysis Results

The results show that the acceptance criteria of Section 4.0 are met. Maximum results are summarized below:

<u>Unit 1</u>

The maximum interaction ratio with respect to the acceptance criteria of AISC N690 is 0.98, which occurs in the ST5WF10.5 bottom chord members.

The stresses in the first panel point of three trusses exceed 1.0 when compared to AISC N690 allowable stresses, however, the maximum strain in these members is 0.15%, which is significantly less than the 1.5% strain limit discussed in Section 4.0.

The maximum liner contact force is 33 kips, which is less than the load limit of 52.8 kips discussed in Section 6.3.

It was concluded from a review of thermal displacements that the thermal analyses of the containment spray pipes that are documented in Ref. 5.25 remain valid.

Unit 2

The maximum interaction ratio with respect to the acceptance criteria of AISC N690 is 0.97, which occurs in the ST9WF32 top chord members.

The stress in the first panel point of one truss exceeds 1.0 when compared to AISC N690 allowable stresses, however, the maximum strain in these members is 0.166%, which is significantly less than the 1.5% strain limit discussed in Section 4.0.

The maximum liner contact force is 41.26 kips, which is less than the load limit of 52.8 kips discussed in Section 6.3.

It was concluded from a review of thermal displacements that the thermal analyses of the containment spray pipes that are documented in Ref. 5.26 remain valid.

6.5.2 Truss Seismic Analysis

The trusses were analyzed for the in-structure seismic demand that was developed as discussed in Section 6.2. The analyses are documented in Ref. 5.10 and 5.11.

6.5.2.1 Analysis Methodology

A review of Ref. 3.11 & 3.12 shows that the clearances between the trusses and the liner vary. During a seismic event, when the truss translates in the direction of the largest gap, it does not contact the liner and is free to displace radially, whereas when the dome truss translates in the direction of a smaller gap, the dome truss interacts with the liner. The interaction with the liner imposes a load on the liner as well as redistributes the loads within the truss itself differently than when it is unrestrained by any contact with the liner.

Three load cases were considered:

1) The truss translates unrestrained by any contact with the liner. This case results in the greatest stresses in the bottom chord. An equivalent static acceleration equal to the spectral demand at the fundamental frequency of the dome truss is applied.

2) The truss is accelerated toward the T1 truss with the smallest clearance between the top chord first panel point at the liner. This case results in the greatest liner contact load for a T1 truss. A nonlinear spring was modeled at the first panel point. The force vs. deflection curve for the nonlinear spring was that developed in Ref. 5.7 as discussed in Section 6.3. Any interaction between the liner and the other trusses is conservatively neglected to maximize the load on truss T1. The spectral demand of the unrestrained support conditions is amplified by an impact factor of 1.33 to account for any increase in loading due to the nonlinear support conditions.

3) The dome truss is accelerated toward the T2 truss with the smallest clearance between the top chord first panel point at the liner. This case results in the greatest liner contact load for a T2 truss. A nonlinear spring was modeled at the first panel point. The force vs. deflection curve for the nonlinear spring was that developed in Ref. 5.7 as discussed in Section 6.3. Any interaction between the liner and the other trusses is conservatively neglected to maximize the load on truss T2. The spectral demand of the unrestrained support conditions is amplified by an impact factor of 1.33 to account for any increase in the truss loading due to the nonlinear support conditions.

An additional load case was performed to determine the acceleration at which an elastic analysis identifies a member stress interaction ratio equal to 1.0. An elastic, response spectra analysis is performed considering boundary conditions without liner contact. The response spectra are scaled down by a factor determined by iteration such that the resulting bounding truss member interaction ratio is nearly equal to 1.0.

The modified Unit 1 dome truss as discussed in Section 6.4.1.1 was analyzed for seismic loads.

As previously analyzed Ref. 5.6, when the dome trusses are free to displace radially, the highest stresses develop in the bottom chords at the intersection with the outermost horizontal braces. Nonlinear plastic hinge sections were included in the models at these locations to allow for elasto-plastic behavior.

For loading cases 2 and 3, to account for the reduced liner clearance when the maximum normal operating temperature inside containment is at 120°F, the liner clearances were reduced by inclusion of thermal growth in the model.

At the minimum normal operating temperature inside containment of 50°F, the clearances with the liner increase due to thermal contraction of the truss, which reduces the number of trusses contacting the liner during a seismic event. In this condition, the dynamic behavior of the truss is closer to loading case 1 for a seismic analysis for the truss translating unrestrained by any contact with the liner.

Loads from attached components were also accounted for, including containment spray support loads obtained from the analyses discussed in Section 6.5.3.

The differential temperature effects due to minimum and maximum design temperatures inside containment cause differential displacements between the dome truss and the containment spray pipes due to differences in material coefficients of thermal expansion and due to dome truss restraint conditions. Ref. 5.10 and 5.11 discuss these effects and conclude that they are negligible.

6.5.2.2 Analysis Results

Unit 1

The maximum interaction ratio with respect to the acceptance criteria of AISC N690 is 0.99, which occurs in the ST5WF10.5 bottom chord members.

The stress in the bottom chord truss members at the outermost horizontal brace exceed 1.0 when compared to AISC N690 allowable stresses, however, the maximum strain in these members is 1.21%, which is less than the 1.5% strain limit discussed in Section 4.0.

The maximum liner contact force is 24.18 kips, which is less than the load limit of 52.8 kips discussed in Section 6.3.

Unit 2

The maximum interaction ratio with respect to the acceptance criteria of AISC N690 is 0.93, which occurs in the bottom chord members.

The stress in the bottom chord truss members at the outermost horizontal brace exceed 1.0 when compared to AISC N690 allowable stresses, however, the maximum strain in these members is 1.22%, which is less than the 1.5% strain limit discussed in Section 4.0.

The maximum liner contact force is 22.18 kips, which is less than the load limit of 52.8 kips discussed in Section 6.3.

6.5.3 Containment Spray Lines Analysis

6.5.3.1 Development of Response Spectra for Analysis of Containment Spray Lines

Time history analyses of the dome truss models of Ref. 5.8 and 5.9 were performed using the BE, LB, and UB TH cases from Ref. 5.13 in each of the two horizontal and vertical directions. TH responses were obtained at each containment spray anchor location on the dome trusses and a response spectrum was generated from each TH.

For each dome truss, the horizontal response spectra for the two horizontal directions at all anchors were enveloped, generating one response spectra for the horizontal direction.

For each dome truss, the vertical response spectra for each containment spray loop were enveloped.

The response spectra damping was 4% as discussed in Section 4.0.

The enveloped response spectra were broadened by 15% in accordance with RG 1.122.

6.5.3.2 Pipe Stress Models

Since only the containment spray lines in the vicinity of the dome trusses are affected by the dynamic responses of the dome trusses, a partial model of each containment spray line was developed and analyzed. The partial models extend beyond the ring sections that are attached to the dome truss to 3 to 5 supports that have a load path directly to the containment wall.

The partial models were developed based on the isometric drawings in Ref. 4.3. The supports are per Ref. 4.6 and 4.7.

To address the effects of local stiffness of the dome truss bottom chord WT members, which are relatively flexible in torsion and weak axis bending, and to which the pipe supports are attached, local stiffness values were included in the pipe stress models. The stiffness values at the pipe supports were determined by applying unit loads in the dome truss models and obtaining the resulting displacements. The stiffness for all 6 degrees of freedom at the anchors were calculated and used in the pipe stress models. For consistency, the vertical stiffness of the trapeze hanger pipe supports attached to the bottom chords of the dome truss were also calculated and included in the pipe stress models.

The anchor stubs (connecting the pipe to the bottom chord of the dome truss) for the Unit 1 lines were modeled as rigid for all degrees of freedom.

The anchor stubs (connecting the pipe to the bottom chord of the dome truss) for the Unit 2 lines were modeled as rigid for all degrees of freedom except for the torsional direction of the anchor supports, which are aligned in the vertical direction. Per Ref. 5.26.4, 5.26.7, and 5.26.10, each anchor support is a W4x13 steel section, which is flexible in the torsional direction (note that per Ref. 4.6.2, 4.6.3, 4.6.8, and 4.6.9, the Unit 1 anchor supports are two C4x5.4 sections welded flange tip to flange tip to form a box section, which is stiffer torsionally relative to the W4x13 section). The torsional stiffness of each Unit 2 anchor is combined in series with the effective torsional stiffness of the bottom chord determined by the dome truss model and applied in the pipe stress model.

All other supports are modeled as rigid except for support 2S249 where, consistent with the current analysis of record (Ref. 5.26.15), its stiffness is included in the pipe stress model.

6.5.3.3 Seismic Input and Analysis

Two levels of seismic demand are used for the pipe system: 1) response spectra applied to the containment wall-mounted supports, which were obtained from the SSI analysis discussed in Section 6.2.4, and 2) response spectra applied to the supports that are attached to the dome trusses, which were those obtained as discussed in Section 6.5.3.1, and which are amplified by the 1.33 impact factor discusses in Section 6.5.2.1. The responses to these two seismic response spectra levels were combined by absolute addition in accordance with Section 2.4 of Ref. 1.12.

Per Section 5.5.4 of Ref. 3.5, the absolute double sum method, identified in RG 1.92 (Ref. 1.13) was used for modal combinations.

In accordance with Section 5.5.6 of Ref. 3.5, 30 Hz was used as the cutoff frequency. Per Section 5.5.9 of Ref. 3.5, Zero Period Acceleration (ZPA) missing mass effects need not be included in the stress analysis if the effects are insignificant. Nevertheless, the missing mass effects were included in the pipe stress analyses.

Seismic anchor movements (SAM) were included in the pipe stress analyses. The SAMs reflected the dome truss displacements from the analysis discussed in Section 6.5.3.1 and account for the elasto-plastic behavior discussed in Section 6.5.2.

Per Section 5.5.5 of Ref. 3.5, "the results from two separate two-directional response spectrum analyses (East-West horizontal combined using SRSS with vertical and North-South horizontal combined using SRSS with vertical) are enveloped".

6.5.3.4 Analysis Results

The analysis of the containment spray lines is documented in Ref. 5.18 and 5.19.

The stresses in the piping were compared to the acceptance criteria discussed in Section 3.4.2. The maximum pipe stress interaction ratio was 0.72 for Unit 1 and 0.79 for Unit 2.

Only the supports that are attached to the dome trusses and two (2) supports on the wall beyond the ring sections that are attached to the dome truss were evaluated. Except for Unit 1 support SI-301R-1-H202, all other supports met the acceptance criteria of Ref. 3.6. The component in support SI-301R-1-H202 that does not meet the acceptance criteria is the U-bolt, which has a load interaction ratio of 1.11. Support SI-301R-1-H202 will be modified to increase the diameter of the U-bolt. Using the GMRS-based seismic input and the ASCE 43-05 damping with limitations per NUREG/CR-6926, the Unit 1 (once modified) and Unit 2 containment spray piping and pipe supports/anchors will conform to Ref. 2.3 and 2.1, respectively.

6.5.4 Other Attached Components

The PACV, HVAC ducts, and lighting that are attached to the dome trusses were evaluated in Ref. 5.10 and 5.11. Stresses were within design allowable values.

6.6 Analysis of Trusses and Attached Components for Lesser Events

The risk informed resolution includes implementation of thermal and seismic operating limits to initiate assessment of the trusses and attached components, for any event exceeding elastic stress limits. Any event reaching or exceeding these operating limit(s) requires Unit shutdown and inspection and/or analysis to ensure the affected structures/components can withstand a subsequent seismic event or a DBA before returning the affected Unit(s) to power operation. The elastic limit stress for steel components was defined as follows: the minimum of 90% of the material minimum yield strength or the allowable stress per AISC N690.

6.6.1 Liner Elastic Capacity

The liner was analyzed to determine its elastic limit capacity for the loading of a single contact point from the trusses. The acceptance criteria for the allowable load on the liner are defined as follows:

The acceptance criteria for the allowable load on the liner are defined as follows:

- The allowable stress limit for the steel liner is defined as 90% of the minimum yield strength.
- The allowable strain limit for the concrete containment wall/shell behind the liner is defined as 0.003 in/in (Ref. 2.2).

The allowable load is defined as the smaller load that reaches one of the two limits defined above.

The analysis is documented in Ref. 5.20 and used the FEM of Ref. 5.7 per the discussion in Section 6.3.

The allowable liner contact force that meets the above criteria is 7.43 kips.

6.6.2 In-Structure Seismic Response at Dome Truss Supports

The in-structure seismic response at the dome truss supports was determined using 40% of the GMRS as input to a SSI analysis. The SSI analysis scaled the THs developed in Ref. 5.12 by 40%. The SSI analysis is documented in Ref. 5.21 and used the same methodology as Ref. 5.13, except for the containment structure damping whereby 3% was used per Table 2 of RG 1.61 for prestressed concrete to reflect a similar dynamic response to an Operating Basis Earthquake demand. 40% of GMRS was chosen, and validated in the seismic analyses (see Section 6.6.4), since at this reduced seismic demand the maximum stress in a truss component reaches elastic limit stress as defined above.

6.6.3 Thermal Limits

The Unit 1 (trimmed at 6 first panel point locations as discussed in Section 6.5.1) and the Unit 2 trusses were analyzed for a thermal load as a result of a DBA. The truss model included non-linear springs at the first panel point to account for the liner stiffness (per Ref. 5.20) and the clearance between the truss and the liner. The analysis is documented in Ref. 5.22. The following are the results of the analysis:

Unit	Thermal Limit	Limiting Component	Maximum Liner Contact Force
1	227°F	Stress in T2 truss bottom chord	7.22 kips
2	236°F	Stress in T1 truss bottom chord	7.40 kips

Since the stresses in attached components are within the elastic range at the DBA temperature of 286°F, they are well within the elastic range at the above temperature levels. The temperatures shown represent an average air temperature experienced by the trusses that results in thermal expansion of all trusses.

6.6.4 Seismic Limits

The Unit 1 (trimmed at 6 first panel point locations as discussed in Section 6.5.1) and the Unit 2 trusses were analyzed for the in-structure seismic demand as discussed in Section 6.6.2. Because the acceptance criteria stress limits were similar to those used for the SSE acceptance criteria, 7% damping for bolted steel structures per Table 1 of RG 1.61 was used.

The Unit 1 truss model included non-linear springs at the first panel point to account for the liner stiffness (per Ref. 5.20) and the clearance between the truss and the liner. The maximum liner contact force from the Unit 1 truss was 75 lbs. Non-linear springs were not used in the analysis of the Unit 2 truss because the analysis results showed no contact with the liner.

The analysis was performed (Ref. 5.23) following the same methodology as discussed in Section 6.5.2.1. The following are the results of the analysis:

Horizontal PGA	Vertical PGA	Limiting Component
0.053	0.045g	Stress in T2 truss bottom chord

As discussed in Ref. 5.23, since the stresses in attached components are within the elastic range at the GMRS input level, they are well within the elastic range at the above PGA levels.

7.0 Summary and Conclusion

Analyses of the existing Units 1 and 2 trusses and attached/adjacent components identified structural nonconformances under design basis thermal and seismic loading with respect to current licensing basis. The resolution of the structural nonconformances is being pursued through a risk-informed approach.

Structural analyses were performed to assess the trusses for DBA thermal loads and develop probabilities of failure vs temperature curves. The following are the conclusions:

- The Unit 1 truss requires limited modification by trimming six first panel point locations to increase clearance with the liner, which results in acceptable truss component stresses and acceptable liner contact load. The probability of failure vs temperature curve of the modified Unit 1 truss is bounded by that of the existing Unit 2 truss.

- No modifications are required for the Unit 2 truss.

The following seismic analyses were performed for an in-structure seismic demand using a soil-structure interaction analysis based on a site-specific ground motion:

- Analyses were performed to show that the Unit 1 truss (with limited modifications as discussed above) and the Unit 2 truss, including attached/adjacent components, maintained structural integrity during the site-specific GMRS-based seismic event. The analyses showed that only containment spray support SI-301R-1-H202 required limited modification.

- Analyses were performed to determine the seismic fragility of the trusses under two different configurations to support the assessment of the ΔCDF per RG 1.174. Below is a summary of the results:

Truss Configuration	Seismic Fragility (Median Capacity)
Unit 1 truss (with limited modifications as discussed above) Unit 2 (no modifications)	0.42g
Unit 1 and 2 trusses with modifications for full code compliance	0.53g

The engineering analyses support the low risk determination in the PRA analysis by concluding that the trusses and attached/adjacent components maintain structural integrity during a design basis seismic or thermal event using the evaluation criteria discussed above, along with the limited modifications discussed above.

The risk informed resolution includes implementation of the following thermal and seismic operating limits to initiate assessment of the trusses and attached components, for any event exceeding elastic stress limits:

Unit	Thermal Limit	Seismic Limit	
		Horizontal PGA	Vertical PGA
1	227°F	0.053g	0.045g
2	236°F		

8.0 References

1. Regulatory Documents
   1.1. NUREG-0800, Standard Review Plan, Section 3.7.1, "Seismic Design Parameters", Rev. 4, December 2014.
   1.2. NUREG-0800, Standard Review Plan, Section 3.7.2, "Seismic System Analysis", Rev. 4, September 2013.
   1.3. NUREG-0800, Standard Review Plan, Section 3.5.3, "Barrier Design Procedures", Rev. 3, March 2007.
   1.4. NUREG-0800, Standard Review Plan, Section 3.8.4, "Other Seismic Category I Structures", Rev. 4, September 2013.
   1.5. Regulatory Guide 1.61, Rev. 1, "Damping Values for Seismic Design of Nuclear Power Plants".
   1.6. Regulatory Guide 1.200, Rev. 2, "An Approach for Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-informed Activities." March 2009.
   1.7. Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)", June, 2007.
   1.8. Regulatory Guide 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion", March 2007.
   1.9. Regulatory Guide 1.122, Rev. 1, "Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components".
   1.10. Regulatory Guide 1.174, Rev. 2, "An Approach for Using Probabilistic Risk Assessment Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis".
   1.11. NUREG CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants".
   1.12. NUREG 1061, Volume 4, December 1984, "Report of the U.S. Nuclear Regulatory Commission Piping Review Committee".
   1.13. NRC Regulatory Guide 1.92, Rev. 3, "Combining Modal Responses and Spatial Components in Seismic Response Analysis".
   1.14. Not Used.
   1.15. NUREG-2115, "Central and Eastern United States Seismic Source Characterization for Nuclear Facilities".
   1.16. NUREG/CR-6728, "Technical Basis for Revisions of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines", October 2001.
   1.17. NUREG/CR-6926, "Evaluation of the Seismic Design Criteria in ASCE/SEI Standard 43-05 for Application to Nuclear Power Plants", March 2007.
   1.18. USNRC NUREG/CR-6906, Containment Integrity Research at Sandia National Laboratories - An Overview, July 2006.
   1.19. Point Beach Nuclear Plant, Units 1 and 2 NRC Integrated Inspection Report 05000266/2014004; 05000301/2014004; and 07200005/2014001 (NRC Accession Number ML14303A355).
2. Codes
   2.1. AISC Manual of Steel Construction, Sixth Edition.
   2.2. ACI 318-63, "Building Code Requirements for Reinforced Concrete".
   2.3. USAS, B31.1 Power Piping Code, 1967 Edition.
   2.4. ASME Boiler and Pressure Vessel Code, Section III, 1977 Issue through Winter 1978 Addenda.
   2.5. ASME Boiler and Pressure Vessel Code, Section III, 1983.
   2.6. ASCE/SEI Standard 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities", American Society of Civil Engineers and Structural Engineering Institute.
   2.7. ANSI/AISC N690-1994(R2004), including Supplement 2, "Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities".
   2.8. ANSI/AISC 341-02, "Seismic Provisions for Structural Steel Buildings".
   2.9. ASTM A36-66, "Standard Specification for Structural Steel".
   2.10. ASTM A442-64, Standard Specification for Carbon Steel Plates with Improved Transition Properties.

2.11. ASME/ANS RA-Sa-2009, "Standard for Level 1/Large Early Release Frequency Probabilistic Risk Assessment for Nuclear Power Plant Applications," Addendum A to RA-S-2008, ASME, New York, NY, American Nuclear Society, La Grange Park, Illinois, February 2009.

3. Station Documents
   - 3.1. Point Beach Nuclear Plant Units 1 & 2 Updated Final Safety Analysis Report, 2014
   - 3.2. Bechtel Spec-0852, Rev. 2, Dated July 1967, "Structural Design Criteria for the Point Beach Nuclear Plant".
   - 3.3. DG-C03, Rev. 3, "Seismic Design Criteria Guideline".
   - 3.4. DG-M02, Rev. 13, "Westinghouse Piping Class Summary".
   - 3.5. DG-M09, Rev. 3, "Design Requirements for Piping Stress Analysis".
   - 3.6. DG-M10, Rev. 3, "Pipe Support Guidelines".
   - 3.7. Bechtel Letter PBB-W-962, dated May 10, 1968, Subj. Point Beach Nuclear Plant Bechtel Job 6118 Containment Dome Truss.
   - 3.8. Westinghouse Letter PBW-B-969, dated February 23, 1968.
   - 3.9. Westinghouse Letter PBW-WMP-718, dated May 17, 1968.
   - 3.10. Westinghouse Letter WEP-97-541, dated December 8, 1997.
   - 3.11. WO 40305152-06, "U1-Containment, Engineering Walkdown"
   - 3.12. WO Task 40316658-01, "2 Cont. / Field Walkdown (ENG, DZ)"
   - 3.13. Specification 6118-C-7, "Specification for Furnishing, Fabrication, Delivery, and Erection of the Containment Structure Liner Plate and Accessory Steel for the Point Beach Nuclear Plant Unit No. 1".
   - 3.14. AR 01677914, "2011 CDBI Inadequate Documentation of Containment Dome Truss".
   - 3.15. Letter NPC-08316 from Sargent & Lundy to Wisconsin Electric Power Company, dated January 29, 1991.
   - 3.16. Specification 6118-M-41, "Specification for Sheetmetal Ductwork, Heating, Ventilating, and Air Conditioning Systems for The Point Beach Nuclear Plant".
4. Drawings
   - 4.1. C-125, Rev. 10, "Containment Structure Liner Support Trusses".
   - 4.2. Dome Truss Shop Drawings
     - 4.2.1. L-23488, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.2. L-23488-A, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.3. L-23488-B, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.4. L-23488-C, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.5. L-23488-D, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.6. L-23488-E, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.7. L-23488-F, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.8. L-23488-G, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.9. L-23488-H, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.10. L-23488-K, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.11. L-23488-M, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.12. L-23488-N, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.13. L-23488-P, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.14. L-23488-T, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.15. L-23488-V, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.16. L-23488-W, Sh. 1, Rev. 0, "U2 Cont. Dome Truss".
     - 4.2.17. L-23488-W, Sh. 2, Rev. 1, "U1 Cont. Dome Truss".
   - 4.3. Containment Spray Isometric Drawings
     - 4.3.1. P-128, Rev. 7, "Discharge from CTMT Spray Pump 1-P14B to Spray Nozzles Inside CTMT Loop B SI-301R-1".
     - 4.3.2. P-129, Rev. 5, "Discharge from CTMT Spray Pump 1-P14A to Spray Nozzles SI-301R-1 Inside CTMT Loop-A".
     - 4.3.3. P-226, Rev. 3, "Discharge from CTMT Spray Pump 2-P14A to Spray Nozzles Ring No. 2 SI-301R-1 Unit 2".
     - 4.3.4. P-229, Rev. 7, "Discharge from CTMT Spray Pump 2-P14B to Spray Nozzle Ring No. 1 SI-301R-1 Unit 2".

- 4.4. PACV Isometric Drawings
  - 4.4.1. P-157, Rev. 3, "Post Accident Containment Venting System 3-A-151R-1 Unit 1".
  - 4.4.2. P-257, Rev. 7, "Post Accident Containment Venting System 3-A-151R-1 Unit 2".
- 4.5. HVAC Ducts
  - 4.5.1. M-123, Rev. 9, "Unit 1 Heating & Ventilation Containment Area 7 Plan Above EL. 86'-0".
  - 4.5.2. M-2123, Rev. 4, "Unit 2 Heating & Ventilation Containment Area 11 Plan Above EL. 86'-0".
- 4.6. Containment Spray Pipe Support Drawings – Unit 1
  - 4.6.1. M-401, Sh. 2, Rev. 1, "Critical Piping Support Assemblies Fabrication, Hanger Guidance & Erection Area 7 – Containment Spray"
  - 4.6.2. P-328, Sheet 1A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-A33 Thru -A35 Point Beach N. P. Unit 1"
  - 4.6.3. P-328, Sheet 1B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-A33 Thru -A35 Point Beach N. P. Unit 1"
  - 4.6.4. P-328, Sheet 16A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H34 Thru -H38, SI-301R-1-H40 Thru -H44, & SI-301R-1-H46 & -H47 Point Beach N. P. Unit 1"
  - 4.6.5. P-328, Sheet 16B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H34 Thru -H38, SI-301R-1-H40 Thru -H44, & SI-301R-1-H46 & -H47 Point Beach N. P. Unit 1"
  - 4.6.6. P-328, Sheet 28A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H62, -H63, & SI-301R-1-SK-1 Point Beach N. P. Unit 1"
  - 4.6.7. P-328 Sheet 28B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H62, -H63, & SI-301R-1-SK-1 Point Beach N. P. Unit 1"
  - 4.6.8. P-329, Sheet 7A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-A36 Thru -A39 Point Beach N. P. Unit 1"
  - 4.6.9. P-329, Sheet 7B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-A36 Thru -A39 Point Beach N. P. Unit 1"
  - 4.6.10. P-329, Sheet 8A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H64 Thru -H67 Point Beach N. P. Unit 1"
  - 4.6.11. P-329, Sheet 8B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H64 Thru -H67 Point Beach N. P. Unit 1"
  - 4.6.12. P-329, Sheet 9A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H49 Thru -H51, SI-301R-1-H53 Thru -H56, & SI-301R-1-H58 Thru -H60 Point Beach N. P. Unit 1"
  - 4.6.13. P-329, Sheet 9B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H49 Thru -H51, SI-301R-1-H53 Thru -H56, & SI-301R-1-H58 Thru -H60 Point Beach N. P. Unit 1"
  - 4.6.14. P-328, Sheet 30, Rev. 1 "Pipe Hanger/Support Detail SI-301R-1-HX5 Point Beach N. P. Unit 1"
  - 4.6.15. P-328, Sheet 31, Rev. 1 "Pipe Hanger/Support Detail SI-301R-1-HX6 Point Beach N. P. Unit 1"
  - 4.6.16. P-328, Sheet 32, Rev. 2 "Pipe Hanger/Support Detail SI-301R-1-S223 Point Beach N. P. Unit 1"
  - 4.6.17. P-328, Sheet 37A, Rev. 2 "Pipe Hanger/Support Detail SI-301R-1-HS15 & -HS16 Point Beach N. P. Unit 1"
  - 4.6.18. P-328, Sheet 37B, Rev. 2 "Pipe Hanger/Support Detail SI-301R-1-HS15 & -HS16 Point Beach N. P. Unit 1"
  - 4.6.19. P-329, Sheet 2, Rev. 2 "Pipe Hanger/Support Detail SI-301R-1-S244 & PA-151R-1-S244 Point Beach N. P. Unit 1"
  - 4.6.20. P-329, Sheet 4A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H202 Point Beach N. P. Unit 1"
  - 4.6.21. P-329, Sheet 4B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H202 Point Beach N. P. Unit 1"
  - 4.6.22. P-329, Sheet 5, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H203 Point Beach N. P. Unit 1"
  - 4.6.23. P-329 Sheet 6, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H204 Point Beach N. P. Unit 1"

- 4.6.24. P-329 Sheet 10A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H201 Point Beach N. P. Unit 1"
- 4.6.25. P-329 Sheet 10B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-H201 Point Beach N. P. Unit 1"

4.7. Containment Spray Pipe Support Drawings – Unit 2

- 4.7.1. M-2401 Sh. 2, Rev. 1, "Critical Piping Support Assemblies Fabrication, Hanger Guidance & Erection Area 11 – Containment Spray"
- 4.7.2. P-426, Sheet 4, Rev. 0 "Pipe Support Reactor Bldg. Disch. From CTMT Spray Pump"
- 4.7.3. P-426, Sheet 10, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H33"
- 4.7.4. P-426, Sheet 11, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H31"
- 4.7.5. P-426, Sheet 12, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H41, SI-301R-1-2H42, & SI-301R-1-2H72 Point Beach N. P. Unit 2"
- 4.7.6. P-426, Sheet 13A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H58 Thru -2H62, SI-301R-1-2H64 Thru -2H68, SI-301R-1-2H70 & SI-301R-1-2H71 Point Beach N. P. Unit 2"
- 4.7.7. P-426, Sheet 13B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H58 Thru -2H62, SI-301R-1-2H64 Thru -2H68, SI-301R-1-2H70 & SI-301R-1-2H71 Point Beach N. P. Unit 2"
- 4.7.8. P-429, Sheet 10, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2S244 Point Beach N. P. Unit 2"
- 4.7.9. P-429, Sheet 12, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2S242 Point Beach N. P. Unit 2"
- 4.7.10. P-429, Sheet 13, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H37 Thru -2H40 Point Beach N. P. Unit 2"
- 4.7.11. P-429, Sheet 14A, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H44 Thru -2H46, SI-301R-1-H48 Thru -2H51, & SI-301R-1-2H53 Thru -2H55 Point Beach N. P. Unit 2"
- 4.7.12. P-429, Sheet 14B, Rev. 0 "Pipe Hanger/Support Detail SI-301R-1-2H44 Thru -2H46, SI-301R-1-H48 Thru -2H51, & SI-301R-1-2H53 Thru -2H55 Point Beach N. P. Unit 2"
- 4.7.13. P-457, Sheet 14, Rev. 0, "Pipe Hanger/Support Detail SI-301R-1-2S250, PA-151R-2S250, Point Beach N. P. Unit 2"

5. Calculations

- 5.1. Bechtel Job 6118 – Vol 8 – Book 44, "Seismic Analysis – Containment".
- 5.2. 6904-15-TR, Rev. 0, "Calc. for Adequacy of Containment Dome Construction Truss."
- 5.3. N-92-085, Rev. 0, "PBNP 90 Day Concrete Strength".
- 5.4. 11Q0060-C-001, Rev. 0, including CCN-001 and CCN-002, "Evaluation of Containment Dome Truss Structural Integrity".
- 5.5. 11Q0060-C-005, Rev. 1, "SSI Analysis of Containment Building for Response Spectra at Supports of Dome Truss".
- 5.6. 11Q0060-C-006, Rev. 1, "Operability Evaluation of Dome Truss Critical Components for Response Spectra Based on SSI Analysis".
- 5.7. 11Q0060-C-021, Rev. 0, "Analysis of Containment Liner for Contact Load from Containment Dome Trusses".
- 5.8. 11Q0060-C-022, Rev. 0, "Thermal Evaluation of Unit 1 Containment Dome Truss in Support of Risk Informed LAR".
- 5.9. 11Q0060-C-023, Rev. 0, "Thermal Evaluation of Unit 2 Containment Dome Truss in Support of Risk Informed LAR".
- 5.10. 11Q0060-C-024, Rev. 0, "Seismic Evaluation of Unit 1 Containment Dome Truss in Support of Risk Informed LAR".
- 5.11. 11Q0060-C-025, Rev. 0, "Seismic Evaluation of Unit 2 Containment Dome Truss in Support of Risk Informed LAR"
- 5.12. 11Q0060-C-026, Rev. 0, "Development of Time Histories Compatible with GMRS Using Real Earthquake Seeds".
- 5.13. 11Q0060-C-027, Rev. 0, "SSI Analysis of Containment Building Using GMRS Input for Response at Dome Truss Supports".
- 5.14. 11Q0060-C-028, Rev. 0, "Seismic Fragility Analysis of Containment Dome Trusses".

5.15. 11Q0060-C-029, Rev. 0, "Verification of KinIntSA".
5.16. 11Q0060-C-030, Rev. 0, "Probability of Failure vs Temperature for Unit 1 Containment Dome Truss in Support of Risk Informed LAR".
5.17. 11Q0060-C-031, Rev. 0, "Probability of Failure vs Temperature for Unit 2 Containment Dome Truss in Support of Risk Informed LAR".
5.18. 11Q0060-C-032, Rev. 0, "Evaluation of Unit 1 Containment Spray Piping under Seismic Loading Using GMRS Input in Support of Risk Informed LAR".
5.19. 11Q0060-C-033, Rev. 0, "Evaluation of Unit 2 Containment Spray Piping under Seismic Loading Using GMRS Input in Support of Risk Informed LAR".
5.20. 11Q0060-C-034, Rev. 0, "Analysis of Containment Liner to Determine Elastic Limit under Contact Load from CDT".
5.21. 11Q0060-C-035, Rev. 0, "SSI Analysis of Containment Building for Lesser Events Using 40% of GMRS Input for Response at Dome Truss Supports".
5.22. 11Q0060-C-036, Rev. 0, "Thermal Evaluation of Units 1 and 2 Containment Dome Trusses for Lesser Events".
5.23. 11Q0060-C-037, Rev. 0, "Seismic Evaluation of Units 1 and 2 Containment Dome Trusses for Lesser Events".
5.24. 11Q0060-C-038, Rev. 0, "Seismic Strength Capacity of Units 1 and 2 Containment Dome Trusses with Modifications to Meet AISC N690 Acceptance Criteria".
5.25. Current Licensing Basis Containment Spray Pipe Stress Analyses – Unit 1
  5.25.1. WE-100092-000, "Containment Spray System Line 3"-SI-301R-1 Between Anchors 1A-34 and 1A-35".
  5.25.2. WE-100092-000-A, "Containment Spray System Line 3"-SI-301R-1 Between Anchors 1A-34 and 1A-35".
  5.25.3. WE-100093-000, "Containment Spray System from Containment Penetration P-55 to Anchors 1A-34 and 1A-35".
  5.25.4. WE-100093-000-D, "Containment Spray System from Containment Penetration P-55 to Anchors 1A-34 and 1A-35".
  5.25.5. WE-100094-000, "Containment Spray System from Containment Penetration P-54 to Anchors 1A-36 and 1A-37".
  5.25.6. WE-100094-000-A, "Containment Spray System from Containment Penetration P-54 to Anchors 1A-36 and 1A-37".
  5.25.7. WE-100094-000-D, "Containment Spray System from Containment Penetration P-54 to Anchors 1A-36 and 1A-37".
  5.25.8. WE-100095-000, "Containment Spray System Line 3"-SI-301R-1 Between Anchors 1A-36 and 1A-37".
5.26. Current Licensing Basis Containment Spray Pipe Stress Analyses – Unit 2
  5.26.1. WE-200069-000, "Containment Spray System Line 2SIG1AC Between Anchors 2A-34 and 2A-35".
  5.26.2. WE-200069-000-A, "Containment Spray System Line 2SIG1AC Between Anchors 2A-34 and 2A-35".
  5.26.3. WE-200069-000-B, "Containment Spray System Line 2SIG1AC Between Anchors 2A-34 and 2A-35".
  5.26.4. WE-200074-000, "6", 4" and 3"-SI-301R-1 Discharge from Containment Penetration P-54 to Spray Nozzles".
  5.26.5. WE-200074-001, "Containment Spray System from Containment Penetration P-54 to Anchors 2A-34 and 2A-35".
  5.26.6. WE-200074-001-C, "Containment Spray System from Containment Penetration P-54 to Anchors 2A-34 and 2A-35".
  5.26.7. WE-200066-000, "Containment Spray from Anchors 2A-36 to Anchor 2A-37".
  5.26.8. WE-200066-000-A, "Containment Spray System Line 2SIG1BC Between Anchors 2A-36 and 2A-37".
  5.26.9. WE-200066-000-B, "Containment Spray System Line 2SIG1BC Between Anchors 2A-36 and 2A-37".

5.26.10. WE-200073-000, "Containment Spray from CTMNT. Pen. P-55 To Anchors 2A-36 and 2A-37".
5.26.11. WE-200073-001, "Containment Spray System from Containment Penetration P-55 to Anchors 2A-36 and 2A-37".
5.26.12. WE-200069S, Rev. 0. "Line 2SIG1AC between Anchors 2A-34 and Anchor 2A-35".
5.26.13. WE-200073-S, Rev. 0. "Containment Spray System from Containment Penetration P-55 to Anchors 2A-36 and 2A-37".
5.26.14. WE-200074-S, Rev. 0. "6", 4", & 3" SI-301R-1 Discharge from Penetration P-54 to Spray Nozzles".
5.26.15. WE-200074-000-B, "6", 4" and 3"-SI-301R-1 Discharge from Containment Penetration P-54 to Spray Nozzles".

6. Reports

6.1. NextEra Energy Point Beach Letter NRC 2014-0024, NextEra Energy Point Beach, LLC Seismic Hazard and Screening Report (CEUS Sites), Response NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, dated March 31, 2014 (NRC Accession Number ML14090A275).
6.2. Point Beach Nuclear Plant, Units 1 and 2 - Staff Assessment of Information Provided Pursuant to Title 10 of the Code of Federal Regulations Part 50, Section 50.54(F), Seismic Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident (Tac Nos. MF3959 And MF3960), dated August 3, 2015 (ML15211A593).
6.3. EPRI 3002004396, July 2015, "High Frequency Program: Application Guidance for Functional Confirmation and Fragility Evaluation".
6.4. NRC Letter, Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, dated March 12, 2012 (NRC Accession Number ML12073A348).
6.5. EPRI 1025287, February 2013, "Seismic Evaluation Guidance Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic".
6.6. EPRI 3002000717, June 2013, "(2004, 2006) Ground-Motion Model (GMM) Review Project" (NRC Accession Number ML13170A385).
6.7. EPRI 1019200, December 2009, Seismic Fragility Applications Guide Update.
6.8. Description and Validation of the Stochastic Ground Motion Model, Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc., Upton, New York 11973, Contract No. 770573, by Silva, W., Abrahamson, N., Toro, G., and Costantino, C.
6.9. 11Q0060-RPT-002, Rev. 1, "Methodology and Criteria to Determine the Strength Capacity of the Point Beach Nuclear Plant Containment Dome Trusses and Attached/Adjacent Components in Support of a Risk-Informed License Amendment Request".
6.10. Brockenbrough, R.L. and Johnston, B.G., "US Steel Design Manual," United States Steel Corporation, Pittsburgh, PA, (1981).
6.11. Galambos, T.V. and Ravindra, M.K., "Properties of Steel for Use in LRFD," ASCE Journal of the Structural Division, Vol. 104, Issue 9, pp. 1459-1468 (1978).
6.12. Beedle, L.S., "Plastic Design of Steel Frames," John Wiley & Sons, Inc., New York, (1958).
6.13. Howland, F.L. and Newmark, N.M., "Static Load Deflection Tests of Beam-Columns," University of Illinois Engineering Experiment Station, College of Engineering, University of Illinois at Urbana-Champaign, Civil Engineering Studies SRS-065 (1953).
6.14. GEI Consultants, Inc. Final Report, "Point Beach Nuclear Plant IPEEE, Two Creeks, Wisconsin", Project 93109, June 1995.
6.15. EPRI NP-6041-SL, Rev. 1, "A Methodology for Assessment of Nuclear Plant Seismic Margin", August 1991.

7. Computer Programs
   7.1. ACS SASSI, Version 3.0 Including Options A, AA and FS - An Advanced Computational Software for 3D Dynamic Analysis Including Soil-Structure Interaction - User Manuals - Revision 3 - March 31, 2015.
   7.2. SUPELM v3.1, Foundation Embedded in Layered Media: Dynamic Stiffnesses and Response to Seismic Waves.
   7.3. EKSSI v3.1, A Program for the Dynamic Analysis of Structures Including Soil-Structure Interaction Effects.