Omaha Public Power District

10 CFR 50.90

LIC-16-0093
October 25, 2016

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

Fort Calhoun Station, Unit No. 1
Renewed Facility Operating License No. DPR-40
NRC Docket No. 50-285

Subject: License Amendment Request (LAR) 16-04; Revise Current Licensing Basis to Use ACI Ultimate Strength Requirements

In accordance with 10 CFR 50.90, the Omaha Public Power District (OPPD) requests an amendment to revise the Renewed Facility Operating License No. DPR-40 for Fort Calhoun Station (FCS), Unit No. 1.

This LAR proposes to revise the FCS Updated Safety Analysis Report (USAR) to change the structural design methodology for the Auxiliary Building at FCS. Specifically, this LAR proposes the following changes:

1. Use the ultimate strength design (USD) method from the ACI 318-63 Code for normal operating/service conditions for future designs and evaluations.
2. Use higher concrete compressive strength values for Class B concrete based on original strength test data.
3. Use higher reinforcing steel yield strength values based on original strength test data.
4. Minor clarifications include adding a definition of control fluids to the dead load section.

The enclosure contains a description of the proposed changes, the supporting technical analysis, and the significant hazards consideration determination. Attachment 1 of the enclosure provides the existing Updated Final Safety Analysis Report (USAR) Section (i.e. Section 5.11) marked-up to show the proposed changes. Attachment 2 of the enclosure provides the retyped (i.e. clean) USAR Section. The proposed changes have been reviewed and approved by the FCS Plant Operations Review Committee (PORC). The amendment will be implemented within 90 days of approval.

OPPD requests review and approval of the proposed license amendment by November 1, 2017, in order to support future decommissioning efforts and activities by resolving design basis issues with the Auxiliary Building. There are no new regulatory commitments contained in this submittal.

In accordance with 10 CFR 50.91, a copy of this application, with attachments, is being provided to the designated State of Nebraska official. If you should have any questions regarding this submittal or require additional information, please contact Mr. Bradley H. Blome, Manager - Site Regulatory Assurance, at 402-533-7270

I declare under penalty of perjury that the foregoing is true and correct. Executed on October 25, 2016.

Respectfully,

Shane Marik

Shane M. Marik
Site Vice President and CNO

SMM/epm

Enclosure: OPPD's Evaluation of the Proposed Change

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OPPD's Evaluation of the Proposed Change

License Amendment Request (LAR) 16-04; Revise Current Licensing Basis to Use ACI Ultimate Strength Requirements

Table of Contents

List of Attachments

1. SUMMARY DESCRIPTION

License amendment request (LAR) 16-04 proposes to revise Renewed Facility Operating License No. DPR-40 to address design basis issues associated with the Auxiliary Building at Fort Calhoun Station (FCS), Unit No. 1. Specifically, this LAR proposes to revise Updated Safety Analysis Report (USAR) Section 5.11, "Structures Other Than Containment" (Reference 6.1) which discusses the design and evaluation of Class I structures at FCS.

USAR Section 5.11 describes the original design criteria for reinforced concrete structures at FCS other than containment. This LAR proposes to apply an alternate design methodology using the ultimate strength design method (USD) provided in the ACI 318-63 Code (Reference 6.3) to new designs or re-evaluations of the existing Auxiliary Building structure at FCS.

The USD method will be applied only to the Auxiliary Building at FCS for normal operating/service conditions. All other structures and foundations, including the Auxiliary Building foundation mat and Spent Fuel Pool, will continue to utilize the most conservative requirements of the current license basis (CLB) obtained by comparing the results of three independent methods of design including the working stress design (WSD) method of the ACI 318-63 Code. As noted in Amendment No. 155 (Reference 6.2), OPPD's structural analysis for the Spent Fuel Pool demonstrates the adequacy and integrity of the pool structure under full fuel loading, thermal loading, and safe shutdown earthquake (SSE) loading conditions and thus no change to the methodology for analyzing the Spent Fuel Pool is proposed.

Prompt NRC approval of this LAR will allow OPPD to reconstitute the Auxiliary Building design in an efficient and cost-effective manner. The proposed changes are an integral part of the analyses necessary to restore the design basis of the Auxiliary Building. Without impacting safety, the changes will increase consistency between the specified loading conditions and allow for the analyses to more accurately reflect the structural response to those loading conditions. Furthermore, with limited clearance inside the Auxiliary Building, it is essential that the number of modifications be limited. Potential modifications may have significant mass and depending on location, their installation may impede access to equipment for plant operation and/or maintenance activities.

The proposed change is for the Auxiliary Building and consists of the following items described in detail in Section 2:

1. Use the USD method from the ACI 318-63 Code for normal operating/service conditions for future designs and evaluations (not applicable to the foundation mat or Spent Fuel Pool).
2. Use higher concrete compressive strength values for Class B concrete based on original strength test data (not applicable to the foundation mat, the Spent Fuel Pool, or exterior walls below the 1007' elevation).
3. Use higher reinforcing steel yield strength values based on original strength test data (not applicable to the foundation mat or Spent Fuel Pool).
4. Minor clarifications include adding a definition of control fluids to the dead load section.

2. DETAILED DESCRIPTION

Planning for the construction of FCS began in 1966. The plant was designed and constructed in accordance with the methodologies of the ACI 318 Building Code, 1963 edition (i.e. the Code of Record (COR), Reference 6.3). Facility Operating License No. DPR-40 was issued to OPPD on August 9, 1973. By letters dated January 9 and April 5, 2002, (References 6.4 and 6.5 respectively), OPPD submitted a license renewal application (LRA) for FCS requesting renewal of the FCS operating license for a period of 20 years beyond the previous expiration of midnight, August 9, 2013. In November 2003, Renewed Facility Operating License No. DPR-40 was issued (Reference 6.6), which expires at midnight, August 9, 2033. On August 25, 2016, OPPD issued LIC-16-0067 certifying that FCS would permanently cease operation by October 24, 2016 (Reference 6.20).

The need to reevaluate portions of the Auxiliary Building was identified as part of an extent of condition review for design issues associated with the Containment Internal Structure (CIS). In 2012, two latent engineering errors were discovered during preparations for a planned extended power uprate of FCS. A detailed review concluded that several concrete beams in the CIS do not meet the current design basis. An operability determination was completed demonstrating that the CIS is operable. A preliminary assessment of the Auxiliary Building was also performed for the extent of condition evaluation which concluded the Auxiliary Building has similar design discrepancies. An operability determination was completed demonstrating that the Auxiliary Building is also operable (Reference 6.23).

The scope of this LAR includes only the Auxiliary Building. After FCS permanently ceases operation in October of 2016, all fuel will be removed from the Containment Building. As a result, High Energy Line Breaks and other accidents that have a significant impact on the CIS are eliminated; therefore, additional design basis analyses for CIS are unnecessary.

The purpose of this LAR is to update and clarify the CLB described in USAR Section 5.11 regarding the methodology used for the analysis of the Auxiliary Building at FCS. For normal service conditions, the USAR requires the Auxiliary Building meet the ACI 318-63 WSD criteria. The USAR specifies the USD method for no loss of function conditions.

This LAR proposes to use the USD method for “Normal Operating/Service Conditions (Operating Basis Earthquake OBE).” This change does not affect “No Loss of Function (Design Basis Earthquake DBE)” and “Accident Conditions (high differential pressure and temperature plus DBE).” Normal/operating service load combinations using USD are developed based on applicable codes and equations. This methodology change will be applied to the Auxiliary Building only. This methodology is approved for use at other nuclear plants (See Section 4.2).

This LAR also proposes material strength changes that will help restore the design basis of the Auxiliary Building and maintain sufficient design margin to ensure that the structure and the safety-related equipment inside it are adequately protected during design basis accidents and natural events.

A description of the Auxiliary Building is as follows:

Auxiliary Building

The Auxiliary Building is a multi-floored, reinforced concrete, Class I structure. From the bottom of the foundation mat to the roof elevation, the structure is of box-type construction with internal bracing provided by vertical concrete walls and horizontal floor slabs. The Spent Fuel Pool is contained within the Auxiliary Building and consists of a stainless steel lined concrete structure. The Control Room is also contained in the Auxiliary Building. The Auxiliary Building masonry walls in the area of safety-related equipment have been reinforced to provide protection for Class I equipment and components located nearby (Reference 6.1).

2.1 Summary of Changes by Item

The following sections discuss each change separately in terms of the CLB, the proposed licensing basis (PLB), the plant structure impacted by the proposed change, design margin impacts, and finally, the consequences of the proposed change.

2.1.1 Concrete normal/service condition design (change from WSD to USD) – See Section 2.2 for Additional Information

The LAR proposes to use the USD method for “Normal Operating/Service Conditions (Operating Basis Earthquake OBE).”

Table 1 – WSD to USD Summary

Current Licensing Basis	Three sets of load combinations are used to evaluate concrete for Class I structures: 1) normal service conditions, 2) no loss-of-function (i.e. design basis loads), and 3) under special cases, accident high pressure and temperature conditions. Normal service condition load combinations are based on WSD methodology. USAR Section 5.11 currently permits the USD methodology only for items 2 and 3.
Proposed Licensing Basis	The PLB change is to normal service conditions where the USD method will be used. The no loss-of-function and special cases load combinations (i.e. Items 2 and 3 above) are unchanged and continue to utilize the USD methodology.
Where Applicable	The Auxiliary Building (not including the foundation mat or Spent Fuel Pool).
Margin	USD Methodology provides adequate margin by requiring the use of load factors and strength reduction factors as identified in ACI 318-63 for concrete and reinforcement steel. USD methodology is covered in ACI 318-63 which is the current license basis and is used for no loss of function and accident conditions. USD methodology for normal service conditions will be used going forward and will meet the requirements of the ACI 318-63 code. The load combination equations are developed for the USD method replacing WSD in the normal service conditions. These are factored load combination equations based on standard use of the factored load combinations (1.4D plus 1.7L) (See Table 5, Note 1). The current license basis is used as the basis for the load combinations that include earthquake and wind loads. The load combination equations are considered to maintain margin per the requirements of the ACI 318-63 Code provisions. These load combinations are similar to those used as part of the design basis for other nuclear power plants.
Consequences	This proposed change when combined with the changes described below is expected to reduce or eliminate the need to install modifications in the Auxiliary Building. This minimizes the potential for harm to SSCs or personnel during installation of potential future modifications.

2.1.2 Use of Increased Concrete Compressive Strength (Based on Original Test Data) – See Section 2.3 for Additional Information

The PLB for concrete strength is to use compressive strength based on original 28-day test data for design basis. This LAR does not propose to use age hardening that occurs beyond 28 days.

Table 2 – Concrete Strength Summary

Current Licensing Basis	Concrete is currently evaluated using the design value from the original construction specifications (i.e. 4000 psi for Class B concrete) (Reference 6.24).
Proposed Licensing Basis	When the plant was built, three (3) test samples were obtained, specifically a set of cylinders were prepared for, each 100 cubic yards or less of concrete placed on a given placement day. The PLB replaces the original specified value of 4000 psi for Class B concrete with 4500 psi based on original 28-day test data which meets the strength testing criteria within ACI 318-63.
Where Applicable	The Auxiliary Building (not including the foundation mat, the Spent Fuel Pool, or exterior walls below 1007' elevation).
Margin	Adequate design margin is maintained in following the criteria for strength testing of concrete as required within ACI 318-63 Section 504. This method is discussed in detail in Section 2.3. The tests completed during construction of the plant in the early 1970s show the concrete is stronger than what was specified in the original design, which indicates higher structural capacity. The PLB takes advantage of the increase in capacity and revises the design basis concrete strength while meeting the testing requirements within ACI 318-63.
Consequences	Use of increased concrete strength increases member design strength resulting in lower interaction ratios and reducing potential modification requirements.

2.1.3 Use of Increased Reinforcing Steel Yield Strength (Use original test data for design basis) – See Section 2.4 for Additional Information

The proposal for reinforcing steel yield strength is to use an increase based on statistical analysis with a 95% confidence level limited to the minimum 95% confidence level for each of the individual bar size data sets.

Table 3 – Reinforcing Steel Yield Strength Summary

Current Licensing Basis	Concrete reinforcement is evaluated using the design value from the original construction specifications (i.e. 40 ksi).
Proposed Licensing Basis	This PLB change replaces the design value from the original construction specifications (40 ksi) with a conservative value derived from original material test data (42 ksi).
Where Applicable	Auxiliary Building (not including the foundation mat or Spent Fuel Pool).
Margin	Adequate design margin is maintained because original test data shows that the mean yield strength with 95% confidence level supports the increase in yield strength.
Consequences	Use of increased rebar strength increases member design strength resulting in lower interaction ratios and reducing potential modification requirements.

2.1.4 Clarification of Hydrostatic Load in the USAR – See Sect. 2.5 for Additional Information

The LAR proposes changes that include adding the definition of controlled fluids.

Table 4 – Hydrostatic Load Clarification Summary

Current Licensing Basis	The CLB has no discussion of hydrostatic load from controlled fluids (i.e. fluid in tanks).
Proposed Licensing Basis	The PLB adopts controlled fluids as a dead load.
Where Applicable	All Class I structures.
Margin	This change is considered to be a clarification that does not impact design margin. It is called out here because it is incorporated in the proposed USAR change but otherwise does not require NRC approval. The PLB treatment of controlled fluids is consistent with industry standards in accounting for the load from fluids stored in vessels such as tanks.
Consequences	This change supports the distribution of loads for controlled fluids and has minimal impact on restoration of the design basis of the Auxiliary Building.

In summary, the changes described in sections 2.1.1 through 2.1.4 along with potential modifications to a limited number of structures will be essential to the restoration of the design basis for the Auxiliary Building. The changes are not universally applied; application is limited to the specific areas described in this LAR.

2.2 Use USD for Normal Operating/Service Conditions

For normal operating/service conditions, the USD factored load combinations and ACI 318-63 USD methods are used. The design loads are not changed; however, definitions for soil pressure (H) and controlled hydrostatic loads and their inclusion into load combinations are added to the USAR for better clarity of the design method.

The strength reduction factors (Φ) which are used in the required ultimate strength capacity (U) for use in normal operating/service conditions remain consistent with those already accepted for use with USD used for the no loss-of-function loading conditions.

The equivalent normal load combinations for USD (PLB) in comparison to WSD (CLB) are as follows:

Table 5 – Load Combinations

Ultimate Strength Design (PLB)	Working Stress Design (CLB)
U = $\frac{1}{\Phi}(1.4\,D + 1.7L + 1.7H)$ (1)	S = D + H + L
U = $\frac{1}{\Phi}(1.0D \pm 0.05D + 1.25L + 1.25W + 1.25H)$ (2)	S = D + L + H + W or E
U = $\frac{1}{\Phi}(1.0D \pm 0.05D + 1.25L + 1.25E + 1.25H)$ (2)	
U = $\frac{1}{\Phi}(1.4D + 1.7H + 1.4F)$ (3)	S = D + H + F

Where:
S = Required section capacity
U = Required ultimate strength capacity
D = Dead load, including internal controlled hydrostatic[4]
L = Live load
H = Soil Load[4]
W = Wind load
E = Design earthquake
F = Flood hydrostatic load to elevation 1007 feet
Φ = Strength reduction factor

(1) The USD load factors for normal operating/service load conditions are consistent with those for Equation 1 in Section 9.2.1 of ACI 349-97 (Reference 6.8) in accordance with the guidance of Section 3.A. of Standard Review Plan (SRP) 3.8.4, Revision 4 (Reference 6.9). Note that these factors are less severe than those required by the ACI 318-63 Code for the USD load combination of Section 1506 equation (15-1). However, the 1.4D and 1.7L load factors were implemented in the ACI 318-71 Code (References 6.10 and 6.13) and remained constant for more than thirty years. In addition, soil pressure (H) (lateral earth pressure or ground water pressure for design of structures below grade) is added to align with editions of ACI 318 issued subsequent to 1963.

(2) The USD load combinations are developed for normal operating/service conditions using the current ultimate strength method USAR equation (i.e. 1/Φ (1.0 D ± 0.05D + 1.25Pc + 1.25E + 1.0Tc)) for the extreme condition, no loss-of-function equation. The equation is modified by replacing the Pc load (differential pressure between compartments resulting from reactor coolant system break) with the live load (L) and replacing the differential temperature (Tc) load with the soil load (H) to include lateral earth pressure, which is aligned with editions of ACI 318 issued after 1963. The load combinations for normal load conditions are similar to the ACI 318-71 Code equation 9-2: U = 0.75 (1.4D + 1.7L + 1.7W)

(3) The USD load combination for normal operating/service load conditions is revised to consider dead load (D) in combination with the hydrostatic flood load (F) and soil pressure (H) to include lateral earth pressure. ACI 318-63 is silent regarding proper consideration of hydrostatic loads and soil pressure. This change is consistent with ACI 349-97 and later revisions of ACI 318.

(4) Soil dynamic pressure and hydrodynamic pressure loading shall be accounted, where applicable in accordance with the current licensing basis.

These changes will be applied to new designs or to re-evaluations of existing reinforced concrete structures of the Auxiliary Building (not including the foundation mat or the Spent Fuel Pool).

For extreme conditions, the no loss-of-function USD load combinations will remain unchanged. However, a clarification to these load combinations includes soil load (H), which was included in design basis calculations. This is not considered a change as it is intended to clarify the license basis.

2.3 Determination of Increased Concrete Compressive Strength Based on Original Test Data

The containment structure was originally designed using Class A concrete (specified as minimum compressive strength 5000 psi). Other Class I structures (i.e. the Auxiliary Building) were originally designed using Class B concrete (specified as minimum compressive strength 4000 psi) (Reference 6.24).

As detailed below, the proposed change will allow reinforced concrete structures in the Auxiliary Building (not including the foundation mat, the Spent Fuel Pool, or exterior walls below 1007' elevation) to be evaluated using concrete compressive strength of 4500 psi for Class B concrete (Reference 6.21). This is based on actual 28-day test data which meets the strength testing requirements of the FCS licensing basis concrete Building Code ACI 318-63 Section 504. This is in lieu of the original specified value of 4000 psi. This is appropriate as original test data shows that the actual 28-day compressive strength of the concrete exceeds the original specified value.

<u>Proposed Methodology for PLB Concrete Strength Determination</u>

The concrete strength for the PLB will be determined through application of FCS licensing basis concrete Building Code ACI 318-63 Section 504(c), which specifies the use of the 28-day laboratory-cured concrete strength test data.

This methodology will be applied for the Auxiliary Building (not including the foundation mat, the Spent Fuel Pool, or exterior walls exposed to soil below 1007' elevation). Specifically, one concrete strength will be established for the Auxiliary Building as follows:

- Documentation of the new design basis concrete strengths will be recorded within a FCS design basis calculation.
- The 28-day test data used will be limited to the areas to which the data applies, i.e. the Auxiliary Building.
- A rolling average will be calculated for five consecutive strength tests for WSD (ACI 318-63 Section 504(c)(1)).
- A rolling average will be calculated for three consecutive strength tests for USD (ACI 318-63 Section 504(c)(2)).
- The 20th percentile of the rolling average strengths will be calculated for WSD (ACI 318-63 Section 504(c)(1)).
- The 10th percentile of the rolling average strengths will be calculated for USD (ACI 318-63 Section 504(c)(2)).

- The minimum of the rolling average strengths for WSD and USD, not more than the 20th and 10th percentile for WSD and USD accordingly, will be used as the new design basis concrete strength.
- If more than the indicated permissive percentage is below the specified strength, the procedures of ACI 214-65 (Reference 6.11) will be employed to determine if the average strength being calculated is adequately in excess of the specified strength (ACI 318-63 Section 504(c)(*)).
- FCS will limit this new design basis concrete strength to no greater than the 95% confidence level of all test data analyzed (additional conservatism beyond ACI 318-63 requirements).

Justification for Proposed Methodology

The increased concrete strength determined with the methodology prescribed in this section satisfy the strength testing requirements of ACI 318-63. As defined in Section 301 of ACI 318-63, "Compressive strength shall be determined by test of standard 6-in. 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."

Per ACI 318-63 Section 504(a), "Specimens made to check… strength of concrete or as a basis for acceptance of concrete shall be made and laboratory-cured in accordance with... (ASTM C31)…. Strength tests shall be made in accordance with… (ASTM C39)." ACI 318-63 Section 504 also states, "Additional test specimens cured entirely under field conditions may be required by the Building Official to check adequacy of curing and protection of the concrete." Test specimens cured entirely under field conditions are not specified for the determination of concrete compressive strength.

In regard to the use of rolling average calculations for data analysis, FCS licensing basis concrete Building Code ACI 318-63 Section 504(c) specifies*, To conform to the requirements of this code*:

[For Working Stress Design]

1. *For structures designed in accordance with Part IV-A of this code, the average of any five consecutive strength tests of the laboratory-cured specimens representing each class of concrete shall be equal to or greater than the specified strength, f_c', and not more than 20 percent of the strength tests shall have values less than the specified strength.*

[For Ultimate Strength Design]

2. *For structures designed in accordance with Part IV-B of this code, and for prestressed structures the average of any three consecutive strength tests of the laboratory-cured specimens representing each class of concrete shall be equal to or greater than the specified strength, f_c', and not more than 10 percent of the strength tests shall have values less than the specified strength.*

Limiting the determined strength to the 95% confidence level is an additional means of establishing a high level of confidence in the new design basis strengths.
In summary, the higher concrete compressive strength value of 4500 psi may be used for activities that require reanalysis of the Auxiliary Building with the following exceptions:

- Auxiliary Building exterior walls below 1007' elevation
- Auxiliary Building foundation mat
- Spent Fuel Pool

The application of increased concrete compressive strength in lieu of original specified design values is not allowed for exterior walls below 1007 feet due to prolonged exposure to excessive moisture. Structure aging management is described in Section 2.7 of this document.

2.4 Determination of Increased Reinforcing Steel Yield Strength Based on Original Test Data

Higher steel yield strength values are proposed for reinforcement of the Auxiliary Building concrete structural members. The increase is limited to the 95% confidence level of the entire data set limited to no greater than the minimum 95% confidence level for each of the individual bar size data sets (Reference 6.22). Section 3.3 of this document provides technical justification for the use of the increased yield strength.

2.5 Clarification of Hydrostatic Load in the USAR Section 5.11

The controlled hydrostatic load is changed from live load to dead load for USD in the definition. The definition for dead load was improved by clarifying that controllable fluids are considered dead load instead of a live load. This is consistent with ACI-349-97 (Reference 6.8).

2.6 Conditions that the Proposed Amendment Will Resolve

2.6.1 Use USD for Normal Operating/Service Conditions

For normal operating/service load cases, FCS's original design calculations used the WSD method, which gives unnecessarily conservative results in comparison to the USD method for the design of reinforced concrete structures. The WSD method gives more conservative results for member capacity compared to using the USD method. The difference in design margin between the two methods varies depending on proportions and types of loads. The ACI 318-63 USD method has been used for the design of reinforced concrete structures with safe, reliable performance for over fifty years, including its use at many other operating nuclear power reactors.

The proposed change only affects the evaluation of normal operating/service conditions. The no loss-of-function, ultimate load combinations for extreme conditions utilizing USD remain unchanged. The no loss-of-function condition evaluations will conservatively maintain the application of differential pressure loads due to a design basis pipe break accident such as a loss-of-coolant accident or main steam line break concurrent with a seismic event. OPPD will continue to maintain the existing design margins for no loss-of-function conditions for extreme conditions at FCS.

The proposed change increases consistency between normal operating/service conditions and no loss-of-function conditions. The current method requiring both WSD and USD calculations is unnecessarily complex, and results in over-designed and over-engineered concrete structures.

The proposed load combination for the USD method to evaluate normal operating/service conditions uses load factors (i.e. 1.4D + 1.7L) consistent with Equation 1 in Section 9.2.1 of ACI 349-97 and in accordance with the guidance of Section 3.A of SRP 3.8.4, Revision 4. Although the ACI 349-97 load factors are lower than those required by the ACI 318-63 Code for the USD load combination of Section 1506, Equation 15-1, the proposed load factors better reflect USD behavior.

Load combinations are developed for normal operating/service conditions using the no loss-of-function USD method equation from USAR Section 5.11
(i.e. $1/\phi$ (1.0D ± 0.05D + 1.25 Pc + 1.25E + 1.0Tc) by removal of differential pressure (Pc) and differential temperature (Tc) loads. These load combinations for normal operating/service load conditions are similar to the ACI 318-71 Code Equation 9-2 below for use with the USD method:

$$U = 0.75(1.4D + 1.7L + 1.7W)$$

The load combination for normal operating/service load conditions using the USD method is a simple conversion to consider loads in combination with the flood load (F).

2.6.2 Determination of Increased Concrete Compressive Strength Based on Original Test Data

The use of the increased concrete compressive strength based on 28-day test data in lieu of the original specified 4000 psi in design re-evaluations will help minimize the potential for modifications in the Auxiliary Building. This includes shear overstress in beams and axial strength of columns for normal operating/service and no loss-of-function loading.

2.6.3 Determination of Increased Reinforcing Steel Yield Strength Based on Original Test Data

The use of increased reinforcing steel yield strength helps resolve bending overstresses in beams. The use of higher yield strength for reinforcing steel is limited to the 95% confidence level of the Certified Material Test Reports. The increase in yield strength maintains design margin and its use in design basis calculations is subject to the limitations previously noted.

2.6.4 Controllable Fluids

The clarifications associated with fluids are consistent with the standard approach to addressing tanks and cavities that contain fluids.

2.7 Evaluation of PLB Change to License Renewal and Aging Management

In Section 3.5 of the LRA (References 6.4 and 6.5), OPPD described its aging management review (AMR) for structural components within the containment and other structures (i.e. Class I, Class II, etc.,) at FCS. The passive, long-lived components in these structures that are subject to an AMR were identified in LRA Tables 2.4.1-1 and 2.4.2.1-1 through 2.4.2.7-1.

OPPD's AMRs included an evaluation of plant-specific and industry operating experience. The plant-specific evaluation included reviews of condition reports and discussions with appropriate site personnel to identify aging effects requiring management. These reviews concluded that the aging effects requiring management, based on FCS operating experience, were consistent with aging effects identified in NUREG-1801, "Generic Aging Lessons Learned (GALL) Report," published July 2001. OPPD's review of industry operating experience included a review of operating experience through 2001. This review concluded that aging effects requiring management based on industry operating experience were consistent with aging effects identified in the GALL Report. OPPD's ongoing review of plant-specific and industry operating experience is conducted in accordance with the FCS operating experience program.

Revision 2 of NUREG-1801 was issued in December 2010. For Condition Report 2013-07171, Westinghouse performed a gap analysis of NUREG-1801 Revision 0 versus Revision 2. Westinghouse noted that the standards referenced in Revision 2 of the GALL do not change the conclusions of the FCS topical reports developed for the LRA that certain aging effects do not require aging management. Acceptable inspections of accessible areas have been performed.

A Structures Monitoring Program (Reference 6.7) is implemented at FCS consisting of periodic inspection and monitoring of structures and structure component supports to ensure that aging degradation leading to loss of intended functions will be detected and that the extent of degradation can be determined. This program is implemented in accordance with NEI 93-01, Revision 2, "Industry Guideline for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants," and Regulatory Guide 1.160, Revision 2, "Monitoring the Effectiveness of Maintenance at Nuclear Power Plants," to satisfy the requirement of 10 CFR 50.65, "Requirements for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants."

Procedure SE-PM-AE-1001, “Auxiliary Building Structural Inspection” is used to monitor the structural condition of the Auxiliary Building. The structure is assessed and results are documented in a manner sufficient to give reasonable assurance that the structure, components, supports, and fasteners are capable of fulfilling their intended design function. This is achieved by performing a visual inspection of accessible surfaces (interior and exterior) of the building (broken down by area/room with reference drawings). The inspection includes all exposed surfaces of the structure, joints and joint material, interfacing structures and material (e.g., abutting soil), embedments, and attached components such as base plates and anchor bolts. In addition, the inspections include a representative sampling of Critical Quality Equipment (CQE) components such as support pedestals; piping and snubber seismic supports; cable tray seismic supports; HVAC seismic supports; a representative sampling of component supports and component support fasteners; pipe supports and equipment anchorage; supports for cable trays, conduits, HVAC ducts, tube track and tubing; anchorage of racks, panels, cabinets; and enclosures for electrical equipment.

Specific industry codes and standards used in the development of the FCS inspection procedure SE-PM-AE-1001 include ACI 349.3R-96, ACI 201.1R-92, ASCE 11-90, RG 1.127 Rev. 1, and Generic Aging Lessons Learned (NUREG-1801, Rev. 0). The Structures Monitoring Program (SMP), which includes SE-PM-AE-1001, is consistent with the requirements of GALL Chapters XI.S5, XI.S6 and applicable criteria in XI.S7. A previous Staff review concluded, per NUREG-1782, that FCS has demonstrated the SMP will effectively manage aging in structures and components in which this program is credited.

The current frequency for performance of SE-PM-AE-1001 is 144 weeks, which meets or exceed the frequency requirements established in ACI 349.3R-96.

Detailed observations for the past three inspections are documented for each elevation of the Auxiliary Building. Findings of the inspections are documented in the FCS Corrective Action Program and/or the completed Structures Monitoring work orders. Typical findings consisted of missing caulking, damaged flashing, damaged coatings, the presence of minor cracks and other minor conditions which were resolved or evaluated and determined to be acceptable as is. To date, no Significant Structural Deterioration has been identified, which is defined as: measurable deterioration which, when compared with past inspections, shows strong evidence of an increase of structural degradation which could affect a building or component's structural integrity or leak tightness.

2.8 Analysis Supporting the License Amendment Changes

Detailed analysis is not yet available to restore the Auxiliary Building to design basis. However, a review of the concrete allowable stresses using WSD compared to the ultimate stresses permitted using USD lends insight into the benefit of using the USD method. A summary of that comparison is shown below in Table 6.

Table 6 – Summary WSD vs. USD Concrete Capacity (Reference 6.3)

Stress Category	WSD Capacity	USD Capacity	Ratio USD:WSD
Bending[1]	$0.45f_c'$	$\Phi 0.85f_c' = 0.765f_c'$	1.70
Shear[2]	$1.1\sqrt{f_c'}$	$2\Phi\sqrt{f_c'} = 1.7\sqrt{f_c'}$	1.55
Compression[3]	$0.25f_c'$	$\Phi 0.85f_c' = 0.595f_c'$	2.38

Notes

1) Stress at extreme compression fiber.
2) Shear stress permitted on an unreinforced web. This does not incorporate additional shear capacity from reinforcing steel.
3) Compression on concrete only.

The highest load factor pertaining to normal operating/service level conditions is 1.7 (i.e. 1.4D + **1.7**L) under the PLB. The increase in member capacity for bending and compression cases benefits from the change to USD because the increase in capacity is equal to or greater than the maximum load factor (recall all load factors are 1.0 under WSD). It is possible member shear interaction ratios may be negatively impacted, although it is anticipated to be unlikely and uncommon because this would require Live Load to be equal to or greater than the total Dead Load on a member (i.e. Average Load Factor = 1.55: where D = L → 1.4D + 1.7L = 1.4D + 1.7D = 1.55D + 1.55D).

When combined with the proposed increases in concrete compressive strength and reinforcing steel yield strength, the use of USD in place of WSD increases member design strength resulting in lower interaction ratios and a reduction in potential modification requirements.

The increase in rebar yield strength requires an increase in bond strength to ensure the concrete can properly support the increase in rebar capacity. Bond strength capacity is calculated in Sections 1301 and 1801 of ACI 318-63. The change in bond strength as it applies to the increase in concrete strength is proportionate to the square root of f'_c since all other terms (i.e. 3.4, 4.8, 6.7 or 9.5 and D for bars conforming to A307) are constants.

The requested increase in concrete compressive strength for the Auxiliary Building corresponds with a 6% increase in concrete bond strength which is greater than the 5% requested increase in rebar yield strength. Therefore, the development of rebar pertaining to bond strength remains adequate under the increased material properties.

The below comparison between proposed and existing material strengths is made based on bond strength capacity as calculated in ACI 318-63.

Ratio of increased bond strength:
$\sqrt{4500}$ psi : $\sqrt{4000}$ psi = 1.06

Ratio of increased yield strength:
42 ksi : 40 ksi = 1.05

2.9 Quality Program During Construction and Quality of Material Tests

The quality program (Reference 6.14) was implemented at the FCS during plant construction. The quality program contained detailed procedures for controlling installation of steel reinforcement and the concrete. Delivery of these materials required testing and records identifying their placement within Class I structures. The records were maintained and transmitted to OPPD (Reference 6.15) and include signoffs for each delivery, steel reinforcement test data, and the location where the steel reinforcement was placed in the Class I structures. Concrete test records are available that accurately show test results and reference the location in the Class I structures where the concrete was placed.

Proper tracking and recording of material locations of concrete pours and location of steel reinforcement was emphasized during plant construction. For example, Reference 6.16 records a discussion confirming the requirements for controlling concrete. Reference 6.16 states:

> *GHD&R* [Gibbs, Hill, Durham, and Richardson] *inspectors are to cover concrete placement continuously and by contact with PKS* [Peter Kiewit Services] *supervision that correct placement is being carried out continuously. This also involves remaining at the pour until satisfied that finishing and curing will be as specified.*

This provides evidence that the installation of concrete and steel reinforcement during plant construction was controlled as documented in the numerous quality records. This gives high confidence that the materials and installation methods used to construct the plant support the changes proposed in this LAR.

3. TECHNICAL EVALUATION

3.1 Use USD Method for Normal Operating/Service Conditions

The change from WSD to USD design methodology for concrete capacities within the same ACI 318-63 Code is an appropriate change; see discussion in Section 1504 of ACI 318-63, Publication SP-10 (Reference 6.12). The materials, construction, and detailing requirements are unchanged. The ACI 318-63 Code included both WSD and USD methods just before the industry began phasing out the application of WSD for reinforced concrete in the 1960s. In 1971, the American Concrete Institute labeled WSD as an alternate design method. In 1999, the American Concrete Institute completely removed the working stress method from the ACI 318 Code.

The current FCS structural design criteria for Class I structures specifies use of ACI 318-63 USD section capacities for extreme (i.e. no loss-of-function) conditions (Reference 6.1). This proposed amendment applies new factored load combinations for normal operating/service conditions using the USD section capacities from the ACI 318-63 Code.

The proposed factored load combinations for normal conditions are required per ACI 318-63 when using USD. The proposed load factors in the first revised load combination (1.4D + 1.7L) will be used for USD to qualify the structure for normal operating/service conditions and are consistent with those in Equation 1 in Section 9.2.1 of ACI 349-97 and are in accordance with the guidance of Section 3.A. of SRP 3.8.4, Revision 4. The ACI 349-97 load factors are lower than those specified by the ACI 318-63 Code in the Section 1506 ultimate load combination equation (15-1). The proposed load factors better reflect USD behavior; see discussion in Section 9.3.1 of ACI 318R-71 (Reference 6.13).

Load combinations are developed for normal operating/service conditions using the no loss-of-function USD method equation from USAR Section 5.11 (i.e. $1/\phi(1.0D \pm 0.05D + 1.25Pc + 1.25E + 1.0Tc)$) by removal of Pc and Tc loads. These load combinations for normal load conditions are similar to the ACI 318-71 Code equation 9-2: $U = 0.75(1.4D + 1.7L + 1.7W)$ for use with the USD method.

The factored load combination for normal load conditions using the USD method are a simple conversion to consider loads in combination with the hydrostatic load (F).

3.2 Determination of Increased Concrete Compressive Strength Based on Original Test Data

The request to implement increased concrete compressive strength while meeting the testing requirements in ACI 318-63 Code is substantiated because test strengths are significantly higher than the original design basis specified 4000 psi minimum compressive strength for Class B concrete. The use of increased concrete compressive strength applies to concrete as defined in Section 301 of ACI 318-63. Section 301 of ACI 318-63 states: “Compressive strength shall be determined by test of standard 6 x 12-in. cylinders made and tested in accordance with American Society for Testing and Materials (ASTM) specifications at 28 days or such earlier age as concrete is to receive its full service load or maximum stress.” By definition, the ACI is concerned with the age at which concrete receives its full load or maximum stress and thereby allowing the full load or maximum stress earlier than 28 days if validated by test data. Therefore, by implication, new or additional loads applied at a later date would be acceptable as long as test data validates the increased compressive strength.

The ACI 318-63 Code is the CLB COR for the design and construction of reinforced concrete structures at FCS. Quality records from concrete pours at FCS as well as original test data are available, and the data supports a higher concrete compressive strength above the minimum specified CLB COR 4000 psi value (Reference 6.21). The use of the actual 28-day properties from the results of compressive tests performed during construction shall be permissible when it can be shown that (1) the samples taken for compressive tests represent the structure being evaluated, and (2) the value selected is derived from a statistical analysis indicating high confidence level. The increased concrete strength is established using analysis of original lab test data meeting the testing requirements of ACI 318-63. The data analysis method used is described in Section 2.3 of this document. Quality records show 912 sample tests were taken throughout the construction of FCS. 903 of 912 sample tests were found; 342 of which apply specifically to the Auxiliary Building.

Application of the increased concrete strength will be limited to areas in the Auxiliary Building as discussed in Section 2.3 of this document.

3.3 Determination of Increased Reinforcing Steel Yield Strength Based on Original Test Data

Quality records show that there were 202 heat code samples used in the construction of the Auxiliary Building. Some of the heat code yield strength values could not be identified, and 184 of the 202 samples are known. Based on 184 samples specifically used for the Auxiliary Building construction, the 95% confidence level is equal to 43.82 ksi for the entire data set. The minimum 95% confidence level for each of the individual bar size data sets is equal to 42.02 ksi for #3 bars. As a result, the current design steel yield strength (i.e. 40 ksi) is increased to 42 ksi with high confidence for the Auxiliary Building (Reference 6.22).

3.4 Clarification of Hydrostatic Load in the USAR Section 5.11

The controlled hydrostatic load is changed from live load to dead load for USD in the definition. The definition for dead load was improved by clarifying that controllable fluids are considered dead load instead of a live load. This is consistent with ACI-349-97.

4. REGULATORY EVALUATION

4.1 Applicable Regulatory Requirements/Criteria

FCS was licensed for construction prior to May 21, 1971, and at that time committed to the draft General Design Criteria (GDC). The draft GDC are contained in Appendix G of the FCS USAR and is similar to 10 CFR 50, Appendix A, *General Design Criteria for Nuclear Power Plants*. The draft GDC most pertinent to this request is USAR Appendix G, Criterion 2, *Performance Standards*. Criteria 2 as described in USAR Appendix G is shown below.

CRITERION 2 - PERFORMANCE STANDARDS

Those systems and components of reactor facilities which are essential to the prevention of accidents which could affect public health and safety or to mitigation of their consequences shall be designed, fabricated, and erected to performance standards that will enable the facility to withstand, without loss of the capability to protect the public, the additional forces that might be imposed by natural phenomena such as earthquakes, tornadoes, flooding conditions, winds, ice and other local site effects. The design bases so established shall reflect: (a) Appropriate consideration for the most severe of these natural phenomena that have been 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.

This criterion is met. The systems and components of the Fort Calhoun Station, Unit No. 1 reactor facility that are essential to the prevention or mitigation of accidents that could affect public health and safety are designed, fabricated, and erected to withstand without loss of capability to protect the public, the additional forces that might be imposed by natural phenomena such as earthquakes, tornadoes, floods, winds, ice and other local site effects.

The containment will be designed for simultaneous stresses produced by the dead load, by 60 psig internal pressure at the associated design temperature, and by the application of forces resulting from an earthquake whose ground motion is 0.08g horizontally and 0.053g vertically. Further, the containment structure will be designed to withstand a sustained wind velocity of 90 mph in combination with the dead load and design internal pressure and temperature conditions. The wind load is based on the highest velocity wind at the site location for 100-year period of recurrence: 90 mph base wind at 30 feet above ground level. Other Class I structures will be designed similarly except that no internal pressure loading is applicable. Class I systems will be designed for their normal operating loads acting concurrently with the earthquake described above.

The containment structure is predicted to withstand without loss of function the simultaneous stresses produced by the dead load, by 75 psig internal pressure and temperature associated with this pressure and by an earthquake whose ground motion is 0.10g horizontally and 0.07g vertically.

The containment structure is predicted to withstand without loss of function 125% of the force corresponding to a 90 mph wind impinging on the building concurrently with the stresses associated with the dead load and 75 psig internal pressure.

With no earthquake or wind acting, the structure is predicted to withstand 90 psig internal pressure without loss of function.

Under each of these conditions, stresses in the structural members will not exceed 0.95 yield.

The facility is designed so that the plant can be safely shutdown and maintained in a safe shutdown condition during a tornado. Design considerations associated with tornadoes are further explained in Section 5.4.7 of the USAR.

Flooding of Fort Calhoun Station, Unit No. 1 is considered highly unlikely. Further information is available in USAR Section 2.7.1.2.

The changes proposed by this LAR will continue to ensure that Criterion 2 is met for the Auxiliary Building.

The USD method is an appropriate method for reinforced concrete design. Quality records list original test data for specific pour locations. This data supports using concrete compressive strength based on actual 28-day test data in lieu of the original specified value of 4,000 psi for Class B concrete. Application of the increased compressive strength will be limited to areas in the Auxiliary Building as discussed in Section 2.3 of this document. The increase in yield strength is supported by statistical analysis of the original test data.

4.2 Precedent

The ACI 318-63 Code was used for the design of nuclear power plants constructed in the era that FCS was built. Operating nuclear power plants licensed to use the ACI 318-63 USD method for normal operating/service load combinations include the following sites:

- Arkansas Nuclear One (ANO)
- Calvert Cliffs
- Turkey Point
- Watts Bar
- Waterford

Furthermore, ANO, Calvert Cliffs, and Waterford were designed using the same proposed 1.25 load factor on operating-basis earthquake (OBE) seismic level loads in their normal operating/service load combinations for the USD method.

The licensing of ACI codes has also evolved at other nuclear plants. For example, the Watts Bar Updated Final Safety Analysis Report (UFSAR) describes the auxiliary-control building as designed in compliance with the ACI 318-63 Code. However, Appendix 3.8E of the Watts Bar UFSAR shows that the current code is the ACI 318-77 Code for the modification and evaluation of existing structures and for design of new features added to existing structures and the design of structures initiated after July 1979.

In January 2003, the NRC issued a memo to D.C. Cook (Reference 6.17) approving the use of 28-day concrete compressive strength tests in design basis calculations.

In March 2005, the NRC approved a license amendment request for D.C. Cook to use reinforcing steel yield strength based on Certified Material Test Report (CMTR) data. Specifically, the licensee (D.C. Cook) limited the increased yield strength to the 95% confidence level (References 6.18 and 6.19).

4.3 No Significant Hazards Consideration

This LAR proposes to revise FCS Unit No. 1, USAR, Section 5.11 "Design Criteria – Class I Structures." The proposed amendment seeks to change structural design criteria and methodology used for the design or re-evaluation of the Auxiliary Building at FCS.

OPPD has evaluated whether or not a significant hazards consideration is involved with the proposed amendment(s) by focusing on the three standards set forth in 10 CFR 50.92, “Issuance of amendment,” as discussed below:

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

   Response: No.

   This LAR revises the methodology used to re-evaluate or design new modifications to the existing Auxiliary Building. All other structures will continue to utilize the current license basis and thus are not affected by this change. The proposed change allows evaluations of the Auxiliary Building to apply the ultimate strength design (USD) method from the ACI 318-63 Code for normal operating/service load combinations.

   The ACI USD method is an accepted industry standard used for the design and analysis of reinforced concrete. A change in the methodology that an analysis uses to verify structure qualifications does not have any impact on the probability of accidents previously evaluated. Designs performed with the ACI USD method will continue to demonstrate that the Auxiliary Building meets industry accepted ACI Code requirements. This LAR does not propose changes to the no loss-of-function loads, loading combinations, or required USD capacity.

   The use of increased concrete strength based on original test data for the areas identified in Section 2.3 of this document and the use of higher steel yield strength maintain adequate structural capacity. As such, these proposed changes do not pose a significant increase in the probability or consequences of an accident previously evaluated because the revised strength values are determined based on actual original test data using a high level of confidence.

   The controlled hydrostatic load is changed from live load to dead load for USD in the definition. This is consistent with ACI-349-97 and therefore does not pose a significant increase in the probability or consequences of an accident previously evaluated.

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

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

   Response: No.

   This LAR proposes no physical change to any plant system, structure, or component. Similarly, no changes to plant operating practices, operating procedures, computer firmware, or computer software are proposed. This LAR does not propose changes to the design loads used to design Class I structures. Application of the new methodology to the design or evaluation of the Auxiliary Building will continue to ensure the Auxiliary Building will adequately house and protect equipment important to safety.

   Calculations that use the ACI USD method for normal operating/service load combinations will continue to demonstrate that the concrete structures meet required design criteria. Use of the increased compressive strength of concrete based on 28-day test data (not age hardening) is permitted by the ACI 318-63 Code and ensures that the concrete structure is capable of performing its design function without alteration or compensatory actions of any kind. A higher steel yield has minimal reduction on design margin. The controlled hydrostatic load is changed from live load to dead load for USD in the definition which is consistent with ACI-349-97.

   The use of these alternative methodologies for qualifying the Auxiliary Building does not have a negative impact on the ability of the structure or its components to house and protect equipment important to safety and thus, does not create the possibility of a new or different kind of accident from any previously evaluated.

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

   Response: No.

   The proposed change is for the design of new modifications or re-analysis of the Auxiliary Building.

   Utilization of the ACI 318-63 Code USD method applies only to the normal operating/service load cases and is already part of the current license basis (CLB) for no loss-of-function load cases. No changes to design basis loads are proposed; therefore, new designs or re-evaluations of the Auxiliary Building shall still prove capable of coping with design basis loads.

   Use of the increased compressive strength of concrete based on 28-day test data is justified and further constrained by limiting its application to areas where the concrete is not exposed to excessive moisture (i.e. exterior walls below 1007' elevation). The use of a higher steel yield is conservatively derived from original test data and has minimal reduction on design margin. The controlled hydrostatic load is changed from live load to dead load for USD in the definition which is consistent with ACI-349-97.

Therefore, the proposed changes do not involve a significant reduction in a margin of safety.

Based on the above, OPPD concludes that the proposed amendment presents no significant hazards consideration under the standards set forth in 10 CFR 50.92(c), and, accordingly, a finding of “no significant hazards consideration” is justified.

4.4 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. ENVIRONMENTAL CONSIDERATION

A review has determined that the proposed amendment would change a requirement with respect to installation or use of a facility component located within the restricted area, as defined in 10 CFR 20, or would change an inspection or surveillance requirement. However, the proposed amendment 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) a significant increase in individual or cumulative occupational radiation exposure.

Accordingly, the proposed amendment 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 need be prepared in connection with the proposed amendment.

6. REFERENCES

6.1 Fort Calhoun Station Unit 1 Updated Safety Analysis Report (USAR) Section 5.11, "Structures Other Than Containment"
6.2 Letter from NRC (S. Bloom) to OPPD (T. L. Patterson), "Fort Calhoun Station, Unit No. 1 – Amendment No. 155 to Facility Operating License No. DPR-40 (TAC No. M85116)," dated August 12, 1993 (NRC-93-0292)
6.3 ACI 318-63, Building Code Requirements for Reinforced Concrete, American Concrete Institute
6.4 Letter from OPPD (W. G. Gates) to NRC (Document Control Desk), "Fort Calhoun Station Unit 1 Application for Renewed Operating License," dated January 9, 2002 (LIC-02-0001) (ML020290333)
6.5 Letter from OPPD (R. P. Clemens) to NRC (Document Control Desk), "Fort Calhoun Station Unit 1 Revised Application for Renewed Operating License," dated April 5, 2002 (LIC-02-0042)
6.6 Letter from NRC (R. K. Anand) to OPPD (R. T. Ridenoure), "Issuance of Renewed Facility Operating License No. DPR-40 Fort Calhoun Station, Unit 1," dated November 4, 2003 (NRC-03-0209) (ML033040033)
6.7 Program Basis Document (PBD)-42, "Structures Monitoring," Revision 4
6.8 ACI 349-97, Code Requirements for Nuclear Safety Related Concrete Structures, American Concrete Institute
6.9 NUREG-0800, Standard Review Plan, Section 3.8.4, "Other Seismic Category I Structures," Rev. 4, 09/2013
6.10 ACI 318-71, Building Code Requirements for Reinforced Concrete, American Concrete Institute
6.11 ACI 214-65, Recommended Practice for Evaluation of Compression Test Results of Field Concrete, American Concrete Institute
6.12 ACI 318-63, Publication SP-10, Commentary on Building Code Requirements for Reinforced Concrete, American Concrete Institute
6.13 ACI 318R-71, Commentary on Building Code Requirements for Reinforced Concrete, American Concrete Institute
6.14 Quality Assurance Program (WIP 008372)
6.15 Quality Assurance Records Transmittal For Batch Documents Number 83-268 (WIP 5932), Dated June 17, 1983
6.16 Letter from J. Woolsey to OPPD (J. Gassman, C. Murphy, F. Wittlinger, C. Mann) "Concrete Control," dated February 5, 1969 (WIP 013439)
6.17 NRC Memorandum ML023290377, "Donald C. Cook, Units 1 & 2, Task Interface Agreement (TIA 2001-15) Re: Evaluation of Containment Structure Conformance to Design-Basis Requirement."
6.18 NRC Memorandum ML052170089, "Cook, Unit 1 - Position Paper on Need for NRC ASME Code Relief."
6.19 NRC Memorandum ML043640141, "D. C. Cook, Units 1 & 2, License Amendment, Re: Request To Use Yield Strength Determined From Measured Material Properties For Reinforcing Bar In Structural Calculations For Control Rod Drive Missile Shields."
6.20 Letter from OPPD (T.J. Burke) to NRC (Document Control Desk), "Certification of Permanent Cessation of Power Operations," dated August 25, 2016 (LIC-16-0067)
6.21 Fort Calhoun Station Calculation FC08499, Evaluation of FCS Concrete Compressive Strength Data

6.22 Fort Calhoun Station Calculation FC08450, Evaluation of Auxiliary Building Concrete Steel Reinforcement CMTR Results
6.23 Fort Calhoun Station Condition Report 2012-04392
6.24 Fort Calhoun Station Original Construction Specification No. 759, Concrete Structures, Containment, Structural Steel and Miscellaneous Facilities for Fort Calhoun Station – Unit No. 1

Fort Calhoun Station, Unit No. 1
Renewed Facility Operating License No. DPR-40

Mark-up of Updated Safety Analysis Report

Section 5.11

USAR-5.11

Structures

Structures Other Than Containment

Rev XX

Safety Classification:

Safety

Usage Level:

Information

Change No.:	
Reason for Change:	
Preparer:	
Issued:	

Fort Calhoun Station

Table of Contents

List of Figures

The following figures are controlled drawings and can be viewed and printed from the listed aperture card.

5.11 Structures Other Than Containment

5.11.1 Classification of Structures

Structures are classified into two categories. Class I and Class II. As described in Appendix F, Class I structures include containment (including all penetrations and air locks, the concrete shield, the liner and the interior structures), the auxiliary building (including the control room, spent fuel pool, safety injection and refueling water storage tank and emergency diesel-generator rooms), and the intake structure up to 1007.5'.

USAR Section 5.11 describes the auxiliary building, intake structure, and the interior structures of containment. The containment building Shell (walls, roof, flat base) is described in Section 5.5.

5.11.2 Description of Class I Structures

The auxiliary building is a Class I structure other than the reactor containment, and is located immediately adjacent to the containment structure.

The foundation mat for the auxiliary building and the containment structure is an integral unit supported on piles driven to bedrock. The piling type and design criteria are presented in Section 5.7.

The auxiliary building is a multi-floored structure of reinforced concrete construction. The building was designed to provide suitable tornado and earthquake protection for the Class 1 equipment and components contained therein. The criteria for this design are given in Section 5.11.3.

The masonry walls in the area of safety-related equipment have been reinforced to provide protection for Class 1 equipment and components nearby.

A section through the engineered safeguards equipment room showing the structural relationship between this room, the auxiliary building and the containment wall, is shown on Figure 5.11-1.

The containment internal structure (CIS) is a reinforced concrete structure consisting of several levels and compartments constructed of beams, slabs and walls supported by reinforced concrete columns located around the periphery of the containment, and the reactor cavity and compartment walls located in the center of containment. The CIS was designed to provide support to major systems and components required for the safe operation of FCS during all operational and outage conditions, including accident mitigation. The CIS is isolated from the containment shell by a shake space which uncouples the response of the shell and dome from that of the internal structure during a seismic event and also permits the distribution and dissipation of any internal pressures during postulated accident events. The criteria for the CIS design are given in Section 5.11.3.

The intake structure is a multi-floored reinforced concrete structure supported by a mat foundation on steel pipe piles driven to bedrock. Major systems and components, both critical quality element (CQE) and non-CQE, that provide water from the Missouri River required for heat removal, are housed within this structure in designated rooms. The building was designed to provide the structural support and environmental protection necessary to ensure the functional integrity of the CQE systems and components under all operational and environmental conditions is maintained. The criteria for this design are given in Section 5.11.3.

5.11.3 Design Criteria - Class I Structures

Class I structures were designed to ensure that their functional integrity under the most extreme environmental loadings, such as tornado or maximum hypothetical earthquake, will not be impaired and thereby, prevent a safe shutdown of the plant.

5.11.3.1 Loading

a. Dead Load (D)

Dead loads included the weight of the structure and other items permanently affixed to it such as equipment, non-structural toppings, partitions, cables, pipes and ducts.

<u>Dead loads also include interior hydrostatic fluid loads which are known and controllable. This type of loading is often sustained over time. This classification is consistent with ACI 349-97 which defines fluid loads as “loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights.”</u>

b. Live Load (L)

Live loads included floor loadings of a magnitude commensurate with their intended use, ice and snow loads on roofs, and impact loads such as may be produced by switchgear, cranes, railroad and equipment handling. Design live loads, with the exception of snow and ice loads, were generally based on temporary or transient loads resulting from the disassembly or replacement of equipment for maintenance purposes. Except for the containment, Class I structures were basically of reinforced concrete box-type construction with internal bracing provided by the vertical concrete interior walls and the horizontal floor slabs. In general, the beams and girders of these structures do not contribute significant lateral shear resistance for the structures, and therefore, in most instances structural elements basically stressed by the floor live loads will not be stressed significantly by the maximum hypothetical earthquake. However, localized areas were investigated and where appropriate, live loads were combined with dead loads and the maximum hypothetical earthquake load. Roof live loads from snow and ice were considered as acting simultaneously with maximum hypothetical earthquake loads.

c. Wind Load (W)

Wind loadings were incorporated as set forth in the ASCE paper No. 3269, Wind Forces on Structures, for the fastest mile of wind, which is 90 mph basic wind 30 feet above ground level at the site for 100 years period of recurrence.

d. Wind Loading due to Tornado (N)

The Class I structures tornado safe shut down analysis has revised design criteria for future evaluations. The methodology for combined loads is maintained. The design basis wind velocity from a tornado event is reduced to a value of 230 mph for the Midwest zones based on studies of tornado winds as defined in Regulatory Guide 1.76, Revision 1.

Class I structures, other than the containment, were originally designed to withstand a tornado with a maximum wind velocity of 300 miles per hour. The wind loads were distributed throughout the structures in accordance with ASCE paper No. 3269, Transactions of the American Society of Civil Engineers, Part II, 1961, utilizing a uniform load throughout the height of the structures.

The grade slab of the auxiliary building was designed to support falling debris that might result from tornado wind speeds in excess of the above structures design wind speed of 300 mph so as to provide additional margin. The emergency diesel generator enclosure and the spent fuel

pool structure were designed to withstand the tornado with a maximum wind velocity of 500 miles per hour, and thus have additional margin beyond the 300 mph basis value.

The 300 mph and 500 mph maximum wind velocities specified in the USAR were considered to be the sums of the translational and rotational components of the tornado.

e. Pressure Loading due to a Differential Pressure (Q)

Class I structures, other than the containment, were designed to withstand a tornado with a maximum wind velocity of 300 miles per hour and a concurrent pressure drop of 3 psi applied in a period of 3 seconds as the tornado passes across the building. This is conservative in comparison to the requirements in Regulatory Guide 1.76, Revision 1. Sufficient venting was provided to prevent the differential pressure, during depressurization, from exceeding a 1.5 psi design value which, when combined with other applicable loads, was determined to be within the allowable load criteria as defined later in this section. Whereas non-vented structures would experience only external depressurization (internal pressures being greater than external pressures) vented structures are subject to external pressurization (internal pressures being lower than external) during the repressurization phase of a tornado. The resulting loads could be more limiting than those of the depressurization phase. The vented structures have, therefore, been subsequently reanalyzed for a complete tornado transient which includes the pressure drop (depressurization) of 3 psi in 3 seconds followed by a low pressure dwell period followed by a recovery pressure rise (repressurization of 3 psi in 3 seconds). The dwell period was sufficient for internal pressures to drop 3 psi prior to repressurization, which results in the most conservative recovery differentials. The transient reanalysis was performed using a suitable dynamic Thermal-Hydraulic analysis code which models the structure as a series of internal volumes connected by various flow paths and vent openings to other volumes and/or boundary conditions. The tornado transient was applied as a time history pressure boundary condition on external flow paths. The structures have been shown to be within design basis allowables for the resultant repressurization differentials combined with other applicable loads thereby demonstrating no loss of function during the repressurization phase of a design basis tornado.

Two cases were considered, during design, in determining vent area requirements. First, a space communicating directly to the outside was treated as a chamber with a sharp edge orifice. The orifice was sized using classical formulae, to give pressure drop of 1.5 psig when flow was fully developed. The flow corresponding to that pressure drop was that required to reduce the pressure in the room by 0.5 psi per second. The criterion developed by this process was that there should be one square foot of vent area for each 1,000 cubic feet of space. This criterion included a margin of safety over the calculation value. For reanalysis, it was conservatively assumed that exterior hinged doors and horizontal concrete relief panels reclose, during repressurization, when air flows reverse in the direction of closure resulting in reduced vent area and higher than 1.5 psid pressure drops.

In many cases, spaces do not communicate directly with the outside, but through another space. For example, the ground floor of the auxiliary building communicates directly to the outside, but the basement communicates indirectly, i.e., through the ground floor. A two stage, iterative model, using the same classical formulae as above, was used to calculate this case for design. The criterion used was that pressure drop across an outside wall should not exceed 1.5 psi, and pressure drop across an interior wall or ceiling should not exceed 1 psi. The calculation was performed on a dynamic basis, i.e., the tornado pressure depression of 0.5 psi per second was assumed to act on initially static conditions. This ramp acted for three seconds, and the ΔP between the basement and the first floor, and between the first floor and the outside was calculated as a function of time. It was found that an opening of one square foot per thousand cubic feet of basement volume was sufficient between the basement and the first floor. Also, an opening in the outside wall of four square feet per thousand cubic feet of first floor volume was sufficient. For reanalysis, it was conservatively assumed that interior hinged doors reclose, during repressurization, if air flow reversed in the direction of closure. This resulted in pressure differentials greater than 1 psi for some interior envelopes.

The vent areas consist primarily of doors and relief panels. These were assumed, for original design, not to be capable of resisting more than approximately 0.5 psi pressure differential. With resistance capability of only one third the design pressure differential, these barriers were expected to open well within the required time. For reanalysis, the existing fire doors installed since original construction were found, from manufacturers data, to have failure ratings greater than 0.5 psid in the open direction. The appropriate values were used for reanalysis. It has been shown that these doors open in time to limit pressure differentials to acceptable values based on the building structures compliance with applicable load limits for no loss of function.

f. Tornado Missile Load

Class I structures were also designed to withstand the spectrum of tornado generated missiles listed in Section 5.8.2.2. The spectrum of tornado generated missiles and the methodology for structural evaluations were updated by Amendment 272.

The methodology uses Regulatory Guide 1.76, Revision 1 and Topical Report BC-TOP-9A, Revision 2 to address protection of SSCs from tornado-generated-missiles at FCS with one exception. The exception regards the potential impact height of an automobile missile where procedural controls prohibit vehicle access to higher surrounding elevations within 0.5 miles of plant structures during periods of increased potential for tornadoes.

g. Seismic Load (E, E')

E = Seismic load from operating basis earthquake (OBE)

E' = Seismic load from maximum hypothetical earthquake (also called Design Basis Earthquake, DBE)

Class I structures were designed for seismic loads as discussed in Appendix F.

Potential seismic loadings were specified as static mechanical loads for the design of the reactor coolant pumps and their drives. These loadings include inertia loadings at the center of gravity of the pump drive assemblies, nozzle loads at the pump suction and discharge and support (hanger) reactions at the pump support lugs. In design calculations for the pump casings, potential seismic loads, in combination with other specified loadings, were evaluated and the calculated stresses limited in accordance with Table 4.2-3.

The seismic input for the internal structure of the reactor vessel, was obtained by "normalizing" the response spectra, Figure F-1 and F-2 (Appendix F) to a ground acceleration equal to the maximum acceleration of the reactor vessel flange.

h. Soil Pressure Load (H)

Load due to lateral earth pressure or ground water pressure for design of structures below grade. Load due to pressure of bulk materials for design of other retention structures.

i. Flood Load (F, F')

F = Flood load to elevation 1007 feet

Hydrostatic load due to lateral pressure of floodwaters to 1007 feet elevation. These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts. Hydrostatic pressure is equal in all directions and acts perpendicular to the surface on which it is applied.

F` = Hydrostatic load to elevation 1014 feet

Class I structures were also designed for the Corps of Engineers estimate of the flood level that might result from the failure of Oahe or Fort Randall dams. The estimated flood level resulting from the failure of a dam coincident with the probable maximum flood is 1014 feet (See Section 2.7.1.2).

5.11.3.2 Operating Basis Load Combinations for Class I Steel Structures

Class I steel structures were designed on the basis of working stress for the following load combinations:

S = D + L + H
S = D + L + H + W or E
S = D + H + F

where:

S = Required section capacity
~~D = Dead load~~
~~L = Live load, including hydrostatic load~~
~~E = Design earthquake~~
~~H = Soil Pressure~~
~~F = Hydrostatic load to elevation 1007 feet~~
~~W = Wind loading~~

5.11.3.3 Design Basis Load Combinations for Class I Steel Structures

Class I steel structures were also designed on the basis of no loss of function for the following load combinations:

S = D + H + E'
S = D + H + L + E'
S = D + N + Q
S = D + 1.25H + F'

where:

S = Required section capacity
~~D = Dead load~~
~~L = Live load, including hydrostatic load~~
~~E' = Maximum hypothetical earthquake~~
~~H = Soil Pressure~~
~~F = Hydrostatic load to elevation 1007 feet~~
~~N = Wind loading as defined by ASCE paper 3269 for a 300 mph tornado wind~~

The AISC Code for Structural Steel, 1963 edition, design methods and allowable stresses were used for steel structures.

5.11.3.4 Operating Basis Load Combinations for Class I Concrete Structures

Class I structures were designed on the basis of working stress for the following load combinations:

S = D + H + L

S = D + L + H + W or E

S = D + H + F

where:

S = Required section capacity
~~D = Dead load~~
~~L = Live load, including hydrostatic load~~
~~W = Wind load~~
~~E = Design earthquake~~
~~F = Hydrostatic load to elevation 1007 feet~~

The ACI Code 318-63 design methods and allowable stresses were used for reinforced concrete.

With the approval of Amendment No. XX, the Auxiliary Building, with the exception of the foundation mat and the Spent Fuel Pool, design criteria changed to implement the ultimate strength design method for normal/operating service conditions for changes and reanalysis using the following load combinations:

$$U = \frac{1}{\Phi}(1.4D + 1.7L + 1.7H)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05\,D + 1.25L + 1.25W + 1.25H)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05\,D + 1.25L + 1.25E + 1.25H)$$

$$U = \frac{1}{\Phi}(1.4D + 1.7H + 1.4F)$$

where:

U = Ultimate strength capacity per the ACI 318-63 Code
Φ = Reduction factors in accordance with the following values and applications:

Φ = 0.90 for concrete in flexure
Φ = 0.90 for mild reinforcing steel in direct tension excluding mechanical or lapped splices
Φ = 0.85 for mild reinforcing steel in direct tension with lapped or mechanical splices
Φ = 0.85 for diagonal tension, bond and anchorage
Φ = 0.70 for tied compression members

The ultimate strength capacity of Class I reinforced concrete structures is determined in accordance with the ultimate strength provisions from the ACI 318-63 Code using the capacity reduction factors, Φ listed above.

5.11.3.5 Design Basis Load Combinations for Class I Concrete Structures

~~Class I structures were also designed on the basis of no loss of function for the following load combinations:~~
Class I concrete structures were designed for no loss of function for the load combinations shown below using the ultimate strength design provisions of the ACI 318-63 Code.

$$U = \frac{1}{\Phi}(1.0D + 1.0H + 1.0E')$$

$$U = \frac{1}{\Phi}(1.0D + 1.0L + 1.0H + 1.0E')$$; Live Load (L) as required.

$$U = \frac{1}{\Phi}(1.0D + 1.0N + 1.0H + 1.0Q)$$

$$U = \frac{1}{\Phi}(1.0D + 1.25H + 1.0F')$$

where: U = Ultimate strength capacity required per the ACI 318-63 Code
~~D = Dead load~~
~~L = Live load~~
~~E' = Seismic load from maximum hypothetical earthquake~~
~~N = Wind loading as defined by ASCE paper 3269 for a 300 mph tornado wind~~
~~Q = Pressure loading due to a differential pressure~~
~~H = Soil Pressure~~
~~F' = Hydrostatic load to elevation 1014 feet~~
~~Φ = Reduction factors as shown in Section 5.11.3.4 above in accordance with the following values and applications:~~
~~Φ = 0.90 for structural steel~~
~~Φ = 0.90 for concrete in flexure~~
~~Φ = 0.90 for mild reinforcing steel in direct tension excluding mechanical or lapped splices~~
~~Φ = 0.85 for mild reinforcing steel in direct tension with lapped or mechanical splices~~
~~Φ = 0.85 for diagonal tension, bond and anchorage~~
~~Φ = 0.70 for tied compression members~~

5.11.3.6 Special Case Load Combinations

a. Load Combinations for Faulted Conditions

The concrete structure within the containment was considered as a Class I structure and was subject to the loads and analysis noted above with the exception of wind and tornado loads. In addition, a transient analysis was made to determine the maximum differential pressure across the interior shielding and structural walls and floors. Openings in the interior concrete walls and floors are provided and grating floors are used wherever possible, without reducing the necessary shielding, to allow pressurization of all compartments with the minimal differential pressure across walls and floors.

In order to provide for the pressure loading resulting from a major break in the reactor coolant system that portion of the concrete structure within the containment surrounding the reactor vessel and reactor coolant system was analyzed and checked on the basis of ultimate strength design methods of ACI Code 318-63 for the factored load combinations given below. The factored load equations are:

$$U = \frac{1}{\Phi}(1.0D \pm 0.05D + 1.5Pc + 1.0Tc)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05D + 1.25Pc + 1.25E + 1.0Tc)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05D + 1.0Pc + 1.0E' + 1.0Tc)$$

where: U, D, E and E' are as defined above, and

Pc = Differential pressure between compartments as a result of a major break in the reactor coolant system.

Tc = Thermal load caused by temperature gradient across the concrete section (generally not applicable to these structures). The capacity reduction factors, Φ, are as given above.

Special steel structures were used around the steam generators for the purpose of limiting the motion of the generator in case a rupture occurs in the reactor coolant piping or in the main steam pipe, or in the feedwater pipe. The energy absorbing members of these structures are hold back rods acting in tension which were designed for strains beyond the elastic limit. The energy due to a pipe break was transformed into strain energy by the yielding of the hold back rods.

b. Load Combinations for Spent Fuel Pool

The spent fuel pool (SFP) structure, including walls, slab and piling, was revisited for the 1994 rerack modification (Ref. 5.13.11). A three-dimensional ANSYS finite element analysis was performed. The design basis and load combinations have been upgraded to those prescribed in the NRC Standard Review Plan (SRP) 3.8.4. After deleting those loads which are not applicable to the SFP structure, the limiting factored load combinations are as follows:

$U = 1.4D + 1.9E$
$U = 0.75\,(1.4D + 1.7T_o + 1.9E)$
$U = D + T_a + E'$
$U = D + T_a + 1.25E$

where:

U = Ultimate strength capacity required
D = Dead load
E = Design earthquake
E' = Maximum hypothetical earthquake
T_a = Abnormal design thermal load
T_o = Normal operating thermal load

The pool is filled with water. The hydrostatic pressure, dead load of racks plus 1083 fuel bundles having conservatively postulated dry weight of 2480 lbs per assembly, water sloshing and convective load, and thermal load were considered. The pool water temperature of 140°F which bounds the normal operating condition was utilized for the analysis. Cracked sections were assumed in the thermal stress analysis. Cracks are usual in reinforced concrete structure. Such credit is permitted by ACI 349-85. The fuel transfer canal, which is next to the spent fuel pool, is assumed to be drained to maximize the loading condition for the spent fuel pool. The calculated loads for the SFP structure, including the walls, slab, and piling, do not exceed the ultimate strength capacity allowable delineated in SRP 3.8.4 and the applicable ACI Code.

A stainless steel liner was provided on the inside face of the pool. This liner plate, due to its ductile nature, will absorb the strain due to the cracking of the concrete in the walls and along with the concrete walls will guarantee tightness of the pool for the full range of credible water temperatures.

5.11.3.7 Codes and Standards

The design of Class I structures, other than the containment, was governed by the then applicable building design codes and standards. In general, those of the American Institute of Steel Construction, the American Concrete Institute, and the American National Standards Institute were followed.

Generally accepted design procedures were used in the development of all structures with modern computerized practices to facilitate the study of all credible combinations of loadings.

Structural steel was designed in accordance with the requirements of the Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, 1963 edition, of the AISC. Elastic theory was the basis of design for all structural steel except for the hold-back bolts at the steam generators.

Reinforced concrete was designed in accordance with the Building Code Requirements for Reinforced Concrete, of the ACI (ACI-318-63) and as stipulated in Section 5.11.3.1a.

5.11.3.8 Concrete Compressive Strength for the Auxiliary Building

Class B concrete was used in the construction of the Auxiliary Building. The compressive concrete strength (f_c') used for the Auxiliary Building during original design was 4000 psi.

A higher concrete design compressive strength value of 4500 psi was established for the Auxiliary Building using statistical analysis of the 28-day test data based on Building Code ACI 318-63 Section 504(c) and limited to no greater than the 95% confidence level of all test data. This value, 4500 psi, was approved in Amendment XX and may be used for activities that require reanalysis of the Auxiliary Building with the following exceptions:

- Auxiliary Building exterior walls below 1007' elevation
- Auxiliary Building foundation mat
- Spent Fuel Pool

5.11.3.9 Steel Reinforcing Yield Strength for the Auxiliary Building

The reinforcing steel yield strength (F_y) used for the Auxiliary Building during original design was 40 ksi.

A higher yield strength value of 42 ksi was established for the Auxiliary Building using statistical analysis of the original test reports for individual bar sizes. The value was limited to the lowest 95% confidence value determined for a bar size data set. This value, 42 ksi, was approved in Amendment XX and may be used for activities that require reanalysis of the Auxiliary Building with the following exceptions:

- Auxiliary Building foundation mat
- Spent Fuel Pool.

5.11.4 Design of Structures - Class II

Class II structures were designed in accordance with conventional practice and on the basis of generally recognized governing codes and criteria such as those of the American Institute of Steel Construction, American Concrete Institute, National Building Code and the American National Standards Institute. The following criteria apply:

a. Dead loads include the weight of the structure and other items permanently affixed to it such as equipment, cables, piping, and ducts.

b. Live loads include floor loadings of a magnitude commensurate with their intended use, ice and snow loads on roofs, and impact loads such as may be produced by equipment, cranes, and handling of equipment.

c. Wind loadings were incorporated as set forth in the National Building Code, 1967 edition, for a moderate windstorm area.

d. Earthquake loads were computed and utilized in accordance with the National Building Code, 1967 edition, as defined in Appendix F, Section F.2.4. These loads were applied to the structure independently of wind loading or horizontal crane impact loading.

e. Horizontal crane impact forces were computed in accordance with the stipulations of the American Institute of Steel Construction, sixth edition.

f. For loading combinations involving wind or earthquake forces, a one-third increase in allowable design stresses was permitted.

g. The design hydrostatic head for Class II structures was assumed to be at elevation 1007'-0". The circulating water tunnels were designed as pressure tunnels with hydrostatic pressures of a magnitude commensurate with their intended use.

For the most part, Class II structures were supported on piling with a compressive load capacity of 90 tons and an uplift capacity of 22.5 tons. Other foundations, separate from the main building, were supported on piling of lesser capacity.

The design of Class II structures was governed by then applicable building design codes and standards such as those of the American Institute of Steel Construction, American Concrete Institute, National Building Code and the American National Standards Institute. Generally accepted design procedures were used.

5.11.5 Visual Weld Acceptance Criteria

Visual weld acceptance criteria for use in structures and supports designed to the requirements of ASIC and AWS D1.1 and other Non-ASME code stamped structures shall be in accordance with AWS D1.1-86 or later revisions, or NCIG-01, Revision 2, titled, Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants. The NCIG-01, Revision 2, document is included as an EPRI document EPRI NP-5380, Volume 1, Research Project Q101, September 1, 1987.

The use of the NCIG-01, Revision 2, acceptance criteria shall be specified in station approved procedures prior to use.

The NCIG-01, Revision 2, has been evaluated by engineering and found to be technically acceptable for use at the Fort Calhoun Station.

Fort Calhoun Station, Unit No. 1
Renewed Facility Operating License No. DPR-40

Clean Updated Safety Analysis Report

Section 5.11

USAR-5.11

Structures

Structures Other Than Containment

Rev XX

Safety Classification:

Safety

Usage Level:

Information

Change No.:	
Reason for Change:	
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Fort Calhoun Station

Table of Contents

List of Figures

The following figures are controlled drawings and can be viewed and printed from the listed aperture card.

5.11 Structures Other Than Containment

5.11.1 Classification of Structures

Structures are classified into two categories. Class I and Class II. As described in Appendix F, Class I structures include containment (including all penetrations and air locks, the concrete shield, the liner and the interior structures), the auxiliary building (including the control room, spent fuel pool, safety injection and refueling water storage tank and emergency diesel-generator rooms), and the intake structure up to 1007.5'.

USAR Section 5.11 describes the auxiliary building, intake structure, and the interior structures of containment. The containment building Shell (walls, roof, flat base) is described in Section 5.5.

5.11.2 Description of Class I Structures

The auxiliary building is a Class I structure other than the reactor containment, and is located immediately adjacent to the containment structure.

The foundation mat for the auxiliary building and the containment structure is an integral unit supported on piles driven to bedrock. The piling type and design criteria are presented in Section 5.7.

The auxiliary building is a multi-floored structure of reinforced concrete construction. The building was designed to provide suitable tornado and earthquake protection for the Class 1 equipment and components contained therein. The criteria for this design are given in Section 5.11.3.

The masonry walls in the area of safety-related equipment have been reinforced to provide protection for Class 1 equipment and components nearby.

A section through the engineered safeguards equipment room showing the structural relationship between this room, the auxiliary building and the containment wall, is shown on Figure 5.11-1.

The containment internal structure (CIS) is a reinforced concrete structure consisting of several levels and compartments constructed of beams, slabs and walls supported by reinforced concrete columns located around the periphery of the containment, and the reactor cavity and compartment walls located in the center of containment. The CIS was designed to provide support to major systems and components required for the safe operation of FCS during all operational and outage conditions, including accident mitigation. The CIS is isolated from the containment shell by a shake space which uncouples the response of the shell and dome from that of the internal structure during a seismic event and also permits the distribution and dissipation of any internal pressures during postulated accident events. The criteria for the CIS design are given in Section 5.11.3.

The intake structure is a multi-floored reinforced concrete structure supported by a mat foundation on steel pipe piles driven to bedrock. Major systems and components, both critical quality element (CQE) and non-CQE, that provide water from the Missouri River required for heat removal, are housed within this structure in designated rooms. The building was designed to provide the structural support and environmental protection necessary to ensure the functional integrity of the CQE systems and components under all operational and environmental conditions is maintained. The criteria for this design are given in Section 5.11.3.

5.11.3 Design Criteria - Class I Structures

Class I structures were designed to ensure that their functional integrity under the most extreme environmental loadings, such as tornado or maximum hypothetical earthquake, will not be impaired and thereby, prevent a safe shutdown of the plant.

5.11.3.1 Loading

a. Dead Load (D)

Dead loads included the weight of the structure and other items permanently affixed to it such as equipment, non-structural toppings, partitions, cables, pipes and ducts.

Dead loads also include interior hydrostatic fluid loads which are known and controllable. This type of loading is often sustained over time. This classification is consistent with ACI 349-97 which defines fluid loads as “loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights.”

b. Live Load (L)

Live loads included floor loadings of a magnitude commensurate with their intended use, ice and snow loads on roofs, and impact loads such as may be produced by switchgear, cranes, railroad and equipment handling. Design live loads, with the exception of snow and ice loads, were generally based on temporary or transient loads resulting from the disassembly or replacement of equipment for maintenance purposes. Except for the containment, Class I structures were basically of reinforced concrete box-type construction with internal bracing provided by the vertical concrete interior walls and the horizontal floor slabs. In general, the beams and girders of these structures do not contribute significant lateral shear resistance for the structures, and therefore, in most instances structural elements basically stressed by the floor live loads will not be stressed significantly by the maximum hypothetical earthquake. However, localized areas were investigated and where appropriate, live loads were combined with dead loads and the maximum hypothetical earthquake load. Roof live loads from snow and ice were considered as acting simultaneously with maximum hypothetical earthquake loads.

c. Wind Load (W)

Wind loadings were incorporated as set forth in the ASCE paper No. 3269, Wind Forces on Structures, for the fastest mile of wind, which is 90 mph basic wind 30 feet above ground level at the site for 100 years period of recurrence.

d. Wind Loading due to Tornado (N)

The Class I structures tornado safe shut down analysis has revised design criteria for future evaluations. The methodology for combined loads is maintained. The design basis wind velocity from a tornado event is reduced to a value of 230 mph for the Midwest zones based on studies of tornado winds as defined in Regulatory Guide 1.76, Revision 1.

Class I structures, other than the containment, were originally designed to withstand a tornado with a maximum wind velocity of 300 miles per hour. The wind loads were distributed throughout the structures in accordance with ASCE paper No. 3269, Transactions of the American Society of Civil Engineers, Part II, 1961, utilizing a uniform load throughout the height of the structures.

The grade slab of the auxiliary building was designed to support falling debris that might result from tornado wind speeds in excess of the above structures design wind speed of 300 mph so as to provide additional margin. The emergency diesel generator enclosure and the spent fuel

pool structure were designed to withstand the tornado with a maximum wind velocity of 500 miles per hour, and thus have additional margin beyond the 300 mph basis value.

The 300 mph and 500 mph maximum wind velocities specified in the USAR were considered to be the sums of the translational and rotational components of the tornado.

e. Pressure Loading due to a Differential Pressure (Q)

Class I structures, other than the containment, were designed to withstand a tornado with a maximum wind velocity of 300 miles per hour and a concurrent pressure drop of 3 psi applied in a period of 3 seconds as the tornado passes across the building. This is conservative in comparison to the requirements in Regulatory Guide 1.76, Revision 1. Sufficient venting was provided to prevent the differential pressure, during depressurization, from exceeding a 1.5 psi design value which, when combined with other applicable loads, was determined to be within the allowable load criteria as defined later in this section. Whereas non-vented structures would experience only external depressurization (internal pressures being greater than external pressures) vented structures are subject to external pressurization (internal pressures being lower than external) during the repressurization phase of a tornado. The resulting loads could be more limiting than those of the depressurization phase. The vented structures have, therefore, been subsequently reanalyzed for a complete tornado transient which includes the pressure drop (depressurization) of 3 psi in 3 seconds followed by a low pressure dwell period followed by a recovery pressure rise (repressurization of 3 psi in 3 seconds). The dwell period was sufficient for internal pressures to drop 3 psi prior to repressurization, which results in the most conservative recovery differentials. The transient reanalysis was performed using a suitable dynamic Thermal-Hydraulic analysis code which models the structure as a series of internal volumes connected by various flow paths and vent openings to other volumes and/or boundary conditions. The tornado transient was applied as a time history pressure boundary condition on external flow paths. The structures have been shown to be within design basis allowables for the resultant repressurization differentials combined with other applicable loads thereby demonstrating no loss of function during the repressurization phase of a design basis tornado.

Two cases were considered, during design, in determining vent area requirements. First, a space communicating directly to the outside was treated as a chamber with a sharp edge orifice. The orifice was sized using classical formulae, to give pressure drop of 1.5 psig when flow was fully developed. The flow corresponding to that pressure drop was that required to reduce the pressure in the room by 0.5 psi per second. The

criterion developed by this process was that there should be one square foot of vent area for each 1,000 cubic feet of space. This criterion included a margin of safety over the calculation value. For reanalysis, it was conservatively assumed that exterior hinged doors and horizontal concrete relief panels reclose, during repressurization, when air flows reverse in the direction of closure resulting in reduced vent area and higher than 1.5 psid pressure drops.

In many cases, spaces do not communicate directly with the outside, but through another space. For example, the ground floor of the auxiliary building communicates directly to the outside, but the basement communicates indirectly, i.e., through the ground floor. A two stage, iterative model, using the same classical formulae as above, was used to calculate this case for design. The criterion used was that pressure drop across an outside wall should not exceed 1.5 psi, and pressure drop across an interior wall or ceiling should not exceed 1 psi. The calculation was performed on a dynamic basis, i.e., the tornado pressure depression of 0.5 psi per second was assumed to act on initially static conditions. This ramp acted for three seconds, and the ΔP between the basement and the first floor, and between the first floor and the outside was calculated as a function of time. It was found that an opening of one square foot per thousand cubic feet of basement volume was sufficient between the basement and the first floor. Also, an opening in the outside wall of four square feet per thousand cubic feet of first floor volume was sufficient. For reanalysis, it was conservatively assumed that interior hinged doors reclose, during repressurization, if air flow reversed in the direction of closure. This resulted in pressure differentials greater than 1 psi for some interior envelopes.

The vent areas consist primarily of doors and relief panels. These were assumed, for original design, not to be capable of resisting more than approximately 0.5 psi pressure differential. With resistance capability of only one third the design pressure differential, these barriers were expected to open well within the required time. For reanalysis, the existing fire doors installed since original construction were found, from manufacturers data, to have failure ratings greater than 0.5 psid in the open direction. The appropriate values were used for reanalysis. It has been shown that these doors open in time to limit pressure differentials to acceptable values based on the building structures compliance with applicable load limits for no loss of function.

f. Tornado Missile Load

Class I structures were also designed to withstand the spectrum of tornado generated missiles listed in Section 5.8.2.2. The spectrum of tornado generated missiles and the methodology for structural evaluations were updated by Amendment 272.

The methodology uses Regulatory Guide 1.76, Revision 1 and Topical Report BC-TOP-9A, Revision 2 to address protection of SSCs from tornado-generated-missiles at FCS with one exception. The exception regards the potential impact height of an automobile missile where procedural controls prohibit vehicle access to higher surrounding elevations within 0.5 miles of plant structures during periods of increased potential for tornadoes.

g. Seismic Load (E, E')

E = Seismic load from operating basis earthquake (OBE)

E' = Seismic load from maximum hypothetical earthquake (also called Design Basis Earthquake, DBE)

Class I structures were designed for seismic loads as discussed in Appendix F.

Potential seismic loadings were specified as static mechanical loads for the design of the reactor coolant pumps and their drives. These loadings include inertia loadings at the center of gravity of the pump drive assemblies, nozzle loads at the pump suction and discharge and support (hanger) reactions at the pump support lugs. In design calculations for the pump casings, potential seismic loads, in combination with other specified loadings, were evaluated and the calculated stresses limited in accordance with Table 4.2-3.

The seismic input for the internal structure of the reactor vessel, was obtained by "normalizing" the response spectra, Figure F-1 and F-2 (Appendix F) to a ground acceleration equal to the maximum acceleration of the reactor vessel flange.

h. Soil Pressure Load (H)

Load due to lateral earth pressure or ground water pressure for design of structures below grade. Load due to pressure of bulk materials for design of other retention structures.

i. Flood Load (F, F')

F = Flood load to elevation 1007 feet

Hydrostatic load due to lateral pressure of floodwaters to 1007 feet elevation. These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts. Hydrostatic pressure is equal in all directions and acts perpendicular to the surface

on which it is applied.

F` = Hydrostatic load to elevation 1014 feet

Class I structures were also designed for the Corps of Engineers estimate of the flood level that might result from the failure of Oahe or Fort Randall dams. The estimated flood level resulting from the failure of a dam coincident with the probable maximum flood is 1014 feet (See Section 2.7.1.2).

5.11.3.2 Operating Basis Load Combinations for Class I Steel Structures

Class I steel structures were designed on the basis of working stress for the following load combinations:

S = D + L + H
S = D + L + H + W or E
S = D + H + F

where:

S = Required section capacity

5.11.3.3 Design Basis Load Combinations for Class I Steel Structures

Class I steel structures were also designed on the basis of no loss of function for the following load combinations:

S = D + H + E’
S = D + H + L + E’
S = D + N + Q
S = D + 1.25H + F’

where:

S = Required section capacity

The AISC Code for Structural Steel, 1963 edition, design methods and allowable stresses were used for steel structures.

5.11.3.4 Operating Basis Load Combinations for Class I Concrete Structures

Class I structures were designed on the basis of working stress for the following load combinations:

$$S = D + H + L$$
$$S = D + L + H + W \text{ or } E$$
$$S = D + H + F$$

where:

S = Required section capacity

The ACI Code 318-63 design methods and allowable stresses were used for reinforced concrete.

With the approval of Amendment No. XX, the Auxiliary Building, with the exception of the foundation mat and the Spent Fuel Pool, design criteria changed to implement the ultimate strength design method for normal/operating service conditions for changes and reanalysis using the following load combinations:

$$U = \frac{1}{\Phi}(1.4D + 1.7L + 1.7H)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05\,D + 1.25L + 1.25W + 1.25H)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05\,D + 1.25L + 1.25E + 1.25H)$$

$$U = \frac{1}{\Phi}(1.4D + 1.7H + 1.4F)$$

where:

U = Ultimate strength capacity per the ACI 318-63 Code
Φ = Reduction factors in accordance with the following values and applications:

- Φ = 0.90 for concrete in flexure
- Φ = 0.90 for mild reinforcing steel in direct tension excluding mechanical or lapped splices
- Φ = 0.85 for mild reinforcing steel in direct tension with lapped or mechanical splices
- Φ = 0.85 for diagonal tension, bond and anchorage
- Φ = 0.70 for tied compression members

The ultimate strength capacity of Class I reinforced concrete structures is determined in accordance with the ultimate strength provisions from the ACI 318-63 Code using the capacity reduction factors, Φ listed above.

5.11.3.5 Design Basis Load Combinations for Class I Concrete Structures

Class I concrete structures were designed for no loss of function for the load combinations shown below using the ultimate strength design provisions of the ACI 318-63 Code.

$$U = \frac{1}{\Phi}(1.0D + 1.0H + 1.0E')$$

$$U = \frac{1}{\Phi}(1.0D + 1.0L + 1.0H + 1.0E')$$; Live Load (L) as required.

$$U = \frac{1}{\Phi}(1.0D + 1.0N + 1.0H + 1.0Q)$$

$$U = \frac{1}{\Phi}(1.0D + 1.25H + 1.0F')$$

where: U = Ultimate strength capacity required per the ACI 318-63 Code

5.11.3.6 Special Case Load Combinations

a. Load Combinations for Faulted Conditions

The concrete structure within the containment was considered as a Class I structure and was subject to the loads and analysis noted above with the exception of wind and tornado loads. In addition, a transient analysis was made to determine the maximum differential pressure across the interior shielding and structural walls and floors. Openings in the interior concrete walls and floors are provided and grating floors are used wherever possible, without reducing the necessary shielding, to allow pressurization of all compartments with the minimal differential pressure across walls and floors.

In order to provide for the pressure loading resulting from a major break in the reactor coolant system that portion of the concrete structure within the containment surrounding the reactor vessel and reactor coolant system was analyzed and checked on the basis of ultimate strength design methods of ACI Code 318-63 for the factored load combinations given below. The factored load equations are:

$$U = \frac{1}{\Phi}(1.0D \pm 0.05D + 1.5Pc + 1.0Tc)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05D + 1.25Pc + 1.25E + 1.0Tc)$$

$$U = \frac{1}{\Phi}(1.0D \pm 0.05D + 1.0Pc + 1.0E' + 1.0Tc)$$

where: U, D, E and E' are as defined above, and

Pc = Differential pressure between compartments as a result of a major break in the reactor coolant system.

Tc = Thermal load caused by temperature gradient across the concrete section (generally not applicable to these structures). The capacity reduction factors, Φ, are as given above.

Special steel structures were used around the steam generators for the purpose of limiting the motion of the generator in case a rupture occurs in the reactor coolant piping or in the main steam pipe, or in the feedwater pipe. The energy absorbing members of these structures are hold back rods acting in tension which were designed for strains beyond the elastic limit. The energy due to a pipe break was transformed into strain energy by the yielding of the hold back rods.

b. Load Combinations for Spent Fuel Pool

The spent fuel pool (SFP) structure, including walls, slab and piling, was revisited for the 1994 rerack modification (Ref. 5.13.11). A three-dimensional ANSYS finite element analysis was performed. The design basis and load combinations have been upgraded to those prescribed in the NRC Standard Review Plan (SRP) 3.8.4. After deleting those loads which are not applicable to the SFP structure, the limiting factored load combinations are as follows:

$U = 1.4D + 1.9E$
$U = 0.75\,(1.4D + 1.7T_o + 1.9E)$
$U = D + T_a + E'$
$U = D + T_a + 1.25E$

where:

U = Ultimate strength capacity required
D = Dead load
E = Design earthquake
E' = Maximum hypothetical earthquake
T_a = Abnormal design thermal load
T_o = Normal operating thermal load

The pool is filled with water. The hydrostatic pressure, dead load of racks plus 1083 fuel bundles having conservatively postulated dry weight of 2480 lbs per assembly, water sloshing and convective load, and thermal load were considered. The pool water temperature of 140°F which bounds the normal operating condition was utilized for the analysis. Cracked sections were assumed in the thermal stress analysis. Cracks are usual in reinforced concrete structure. Such credit is permitted by ACI 349-85. The fuel transfer canal, which is next to the spent fuel pool, is assumed to be drained to maximize the loading condition for the spent fuel pool. The calculated loads for the SFP structure, including the walls, slab, and piling, do not exceed the ultimate strength capacity allowable delineated in SRP 3.8.4 and the applicable ACI Code.

A stainless steel liner was provided on the inside face of the pool. This liner plate, due to its ductile nature, will absorb the strain due to the cracking of the concrete in the walls and along with the concrete walls will guarantee tightness of the pool for the full range of credible water temperatures.

5.11.3.7 Codes and Standards

The design of Class I structures, other than the containment, was governed by the then applicable building design codes and standards. In general, those of the American Institute of Steel Construction, the American Concrete Institute, and the American National Standards Institute were followed.

Generally accepted design procedures were used in the development of all structures with modern computerized practices to facilitate the study of all credible combinations of loadings.

Structural steel was designed in accordance with the requirements of the Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, 1963 edition, of the AISC. Elastic theory was the basis of design for all structural steel except for the hold-back bolts at the steam generators.

Reinforced concrete was designed in accordance with the Building Code Requirements for Reinforced Concrete, of the ACI (ACI-318-63) and as stipulated in Section 5.11.3.1a.

5.11.3.8 Concrete Compressive Strength for the Auxiliary Building

Class B concrete was used in the construction of the Auxiliary Building. The compressive concrete strength (f_c') used for the Auxiliary Building during original design was 4000 psi.

A higher concrete design compressive strength value of 4500 psi was established for the Auxiliary Building using statistical analysis of the 28-day test data based on Building Code ACI 318-63 Section 504(c) and limited to no greater than the 95% confidence level of all test data. This value, 4500 psi, was approved in Amendment XX and may be used for activities that require reanalysis of the Auxiliary Building with the following exceptions:

- Auxiliary Building exterior walls below 1007' elevation
- Auxiliary Building foundation mat
- Spent Fuel Pool

5.11.3.9 Steel Reinforcing Yield Strength for the Auxiliary Building

The reinforcing steel yield strength (F_y) used for the Auxiliary Building during original design was 40 ksi.

A higher yield strength value of 42 ksi was established for the Auxiliary Building using statistical analysis of the original test reports for individual bar sizes. The value was limited to the lowest 95% confidence value determined for a bar size data set. This value, 42 ksi, was approved in Amendment XX and may be used for activities that require reanalysis of the Auxiliary Building with the following exceptions:

- Auxiliary Building foundation mat
- Spent Fuel Pool.

5.11.4 Design of Structures - Class II

Class II structures were designed in accordance with conventional practice and on the basis of generally recognized governing codes and criteria such as those of the American Institute of Steel Construction, American Concrete Institute, National Building Code and the American National Standards Institute. The following criteria apply:

a. Dead loads include the weight of the structure and other items permanently affixed to it such as equipment, cables, piping, and ducts.

b. Live loads include floor loadings of a magnitude commensurate with their intended use, ice and snow loads on roofs, and impact loads such as may be produced by equipment, cranes, and handling of equipment.

c. Wind loadings were incorporated as set forth in the National Building Code, 1967 edition, for a moderate windstorm area.

d. Earthquake loads were computed and utilized in accordance with the National Building Code, 1967 edition, as defined in Appendix F, Section F.2.4. These loads were applied to the structure independently of wind loading or horizontal crane impact loading.

e. Horizontal crane impact forces were computed in accordance with the stipulations of the American Institute of Steel Construction, sixth edition.

f. For loading combinations involving wind or earthquake forces, a one-third increase in allowable design stresses was permitted.

g. The design hydrostatic head for Class II structures was assumed to be at elevation 1007'-0". The circulating water tunnels were designed as pressure tunnels with hydrostatic pressures of a magnitude commensurate with their intended use.

For the most part, Class II structures were supported on piling with a compressive load capacity of 90 tons and an uplift capacity of 22.5 tons. Other foundations, separate from the main building, were supported on piling of lesser capacity.

The design of Class II structures was governed by then applicable building design codes and standards such as those of the American Institute of Steel Construction, American Concrete Institute, National Building Code and the American National Standards Institute. Generally accepted design procedures were used.

5.11.5 Visual Weld Acceptance Criteria

Visual weld acceptance criteria for use in structures and supports designed to the requirements of ASIC and AWS D1.1 and other Non-ASME code stamped structures shall be in accordance with AWS D1.1-86 or later revisions, or NCIG-01, Revision 2, titled, Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants. The NCIG-01, Revision 2, document is included as an EPRI document EPRI NP-5380, Volume 1, Research Project Q101, September 1, 1987.

The use of the NCIG-01, Revision 2, acceptance criteria shall be specified in station approved procedures prior to use.

The NCIG-01, Revision 2, has been evaluated by engineering and found to be technically acceptable for use at the Fort Calhoun Station.