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LIC-15-0142

10 CFR 50.90

December 23, 2015

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: Supplement of License Amendment Request 15-03; Revise Current Licensing Basis to Use ACI Ultimate Strength Requirements

References:
1. Letter from OPPD (L. P. Cortopassi) to NRC (Document Control Desk), "License Amendment Request 15-03; Revise Current Licensing Basis to Use ACI Ultimate Strength Requirements," dated August 31, 2015 (LIC-15-0077) (ML15243A167)
2. Letter from NRC (C. F. Lyon) to OPPD (L. P. Cortopassi), "Fort Calhoun Station, Unit No. 1 - Supplemental Information Needed for Acceptance of Requested Licensing Action RE: Revise Current Licensing Basis to use American Concrete Institute Ultimate Strength Requirements (CAC NO. MF6676)," dated December 15, 2015 (NRC-15-104) (ML15341A224)

In Reference 2, the U.S. Nuclear Regulatory Commission (NRC) notified the Omaha Public Power District (OPPD) that OPPD's License Amendment Request (LAR) (Reference 1) proposing to revise the current licensing basis to use American Concrete Institute (ACI) ultimate strength requirements was not acceptable as additional information was needed. As noted in Reference 2, and subsequent teleconferences with the NRC, OPPD is providing this supplement to supersede the Reference 1 LAR in its entirety. The enclosure to this letter has been revised to address the information request of Reference 2.

This License Amendment Request (LAR) proposes to revise the FCS Updated Safety Analysis Report (USAR) to change the structural design methodology for Class I structures at FCS with several exceptions. The exceptions are the containment structure (cylinder, dome, and base mat), the spent fuel pool, and the foundation mats. No change to the current licensing basis code of record is proposed for those structures. Specifically, this LAR proposes the following changes:

1. Replace the working stress design (WSD) method with the ultimate strength design (USD) method from the ACI 318-63 Code for normal operating/service conditions associated with Class I structures other than the containment structure, the spent fuel pool, and the foundation mats.

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2. Replace the original specified value (i.e. 4000 psi for Class B concrete) with the actual compressive design strength based on original 28-day test data which meets the strength testing criteria within ACI 318-63 (i.e. not age hardening).
3. Use higher reinforcing steel yield strength values for the containment internal structure (CIS) that includes the reactor cavity and compartment (RC&C) and CIS beams, slabs and columns.
4. Use the limit design method for evaluating the concrete in the RC&C walls including dynamic increase factors (DIF) for impulsive loads.
5. 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 USAR Sections (i.e. Sections 5.2 & 5.11) marked-up to show the proposed changes. Attachment 2 of the enclosure provides the retyped (i.e. clean) USAR Sections. The proposed changes have been reviewed and approved by the Fort Calhoun Station Plant Operations Review Committee (PORC) and by the Nuclear Safety Review Board (NSRB). The amendment will be implemented within 90 days of approval.

Although this LAR is neither exigent nor emergency, prompt staff review is requested with approval by June 1, 2016 in order to resolve design basis issues with Class I structures. 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. Bill R. Hansher at (402) 533-6894.

I declare under penalty of perjury that the foregoing is true and correct. Executed on December 23, 2015.

Respectfully,

Louis P. Cortopassi
Site Vice President and CNO

LPC/JAC/mle

Enclosure: OPPD's Evaluation of the Proposed Change

c: M. L. Dapas, NRC Regional Administrator, Region IV
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OPPD's Evaluation of the Proposed Change

Supplement of License Amendment Request (LAR) 15-03; Revise Current Licensing Basis to Use ACI Ultimate Strength Requirements

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2. Retyped ("Clean") USAR Section 5.2 and USAR Section 5.11

1.0 SUMMARY DESCRIPTION

License amendment request (LAR) 15-03 proposes to revise Renewed Facility Operating License No. DPR-40 to address design basis issues associated with Class I structures 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), and USAR Section 5.2, "Materials of Construction" (Reference 6.2), which discuss the design and evaluation of Class I structures at FCS. USAR Section 5.5, "Containment Design Criteria" (Reference 6.3) also contains information describing the original design criteria for Class I structures at FCS; however, no changes to USAR Section 5.5 are necessary or proposed.

USAR Sections 5.5 and 5.11, describe the original design criteria for reinforced concrete structures at FCS. This LAR proposes to apply the alternate design methodology using the ultimate strength design method (USD) provided in the ACI 318-63 Code to new designs or re-evaluations of existing Class I structures at FCS.

The USD method will be applied to all Class I structures at FCS except the containment structure, the spent fuel pool (SFP), and the foundation. The containment structure, SFP, and the foundation mats 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 6.4), OPPD's structural analysis for the SFP 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 SFP is proposed.

Prompt NRC approval of this LAR will allow OPPD to proceed in an optimum, safe, and effective manner. The proposed methodology changes are an integral part of the analyses necessary to restore the design basis of the containment internal structures. The analyses ensure that any physical modifications necessary to resolve this issue are properly engineered and constructed. With limited clearance inside the containment building, it is essential that any additional structures placed in the building be optimized and provide tangible safety benefits. These structures will have significant mass and the act of maneuvering them into position has the potential to harm plant structures, workers, and/or the nuclear steam supply system (NSSS) or other safety-related equipment if an accident were to occur. New structures may also impede access to plant equipment necessary for plant operation and/or maintenance activities particularly during refueling outages.

The proposed change consists of the following items described in detail in Section 2.0:

1. Replace the working stress design (WSD) method with the ultimate strength design (USD) method from the ACI 318-63 Code for normal operating/service conditions associated with Class I structures other than the containment structure, the spent fuel pool, and the foundation mats.

2. Replace the original specified value (i.e. 4000 psi for Class B concrete) with the actual compressive design strength based on original 28-day test data which meets the strength testing criteria within ACI 318-63 (i.e. not age hardening).

3. Use higher reinforcing steel yield strength values for the containment internal structure (CIS) that includes the reactor cavity and compartment (RC&C) and CIS beams, slabs and columns.

4. Use the limit design method for evaluating the concrete in the RC&C walls including dynamic increase factors (DIF) for impulsive loads.

5. Minor clarifications include adding a definition of control fluids to the dead load section.

2.0 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 6.5)). Facility Operating License No. DPR-40 was issued to the Omaha Public Power District (OPPD) on August 9, 1973. By letters dated January 9 and April 5, 2002, (References 6.6 and 6.7 respectively), the Omaha Public Power District (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 6.8), which expires at midnight, August 9, 2033.

In 2012, two latent engineering errors were discovered during preparations for a planned extended power uprate of FCS. The first discrepancy is in calculation FC01421 where the steel reinforcement area in the calculation (design basis) is higher than the steel reinforcement area installed per the construction shop drawing. The second discrepancy is the reaction from beam B-64 onto beam B-59. The calculation contains an error where the reaction on B-59 was low by a factor of 3.6. A detailed extent of condition 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. The operability determination was supported by several calculations that specified the criteria and methodology, performed a thorough review of dead loads, and executed a complete modeling analysis. As noted in Reference 6.9, the operability determination was conducted with computer software assessing the capability of the CIS with the USD method.

The purpose of this LAR is to update and clarify the CLB described in USAR Sections 5.2 and 5.11 regarding the codes and standards used for the analysis of Class I structures at FCS. For normal service conditions, the USAR requires the CIS to meet the ACI 318-63 working stress design criteria. The USAR specifies the USD method for no loss of function conditions. This LAR proposes methodology changes that will restore the design basis of the Class I structures and maintain sufficient design margin to ensure that the structures and the safety-related equipment inside them are adequately protected during design basis accidents and natural events. Each of the Class I structures has been evaluated and is operable.

Overall characterization of the changes are adjustments to the CLB for application of the COR. These adjustments include using USD replacing WSD and use of actual strengths for applicable materials within the COR. The changes include adding state of the art methods where the COR is silent.

A description of the Class I structures discussed within this LAR is as follows:

Containment

The containment building is a partially prestressed, reinforced concrete, Class I structure composed of a cylindrical wall and domed roof (i.e. shell) and a foundation mat. The foundation mat is common to both the containment structure and the auxiliary building and is supported on steel piles driven to bedrock. The mat incorporates a depressed center portion for the reactor vessel. The containment building has an internal carbon steel liner.

The containment building houses a substantial amount of safety-related and non-safety related mechanical and electrical equipment and there are many mechanical piping and electrical penetrations through the cylindrical wall.

Containment Internal Structure

The CIS is an independent, multi-story, reinforced concrete structure located inside the containment structure. The CIS is isolated from the containment structure by a shake space that permits the distribution and dissipation of any internal differential pressure during postulated accident events. The CIS is composed of 269 structural elements (135 reinforced concrete beams, 32 reinforced concrete column sections, and 102 reinforced concrete slabs) with an internal core consisting of the RC&C that physically support safety related components – including the reactor coolant system, emergency core cooling system piping, and the reactor components.

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.

Intake Structure

The intake structure is a multi-floored Class I structure. From the bottom of the foundation mat to 7 feet above the operating floor, the structure is a box-type reinforced concrete structure with internal bracing provided by concrete walls and floor slabs. The mat foundation is supported on steel pipe piles driven to bedrock. Above the reinforced concrete structure to the roof, the structure is a braced steel frame clad with aggregate resin panels. The multi-layered built-up roof is supported by metal decking spanning between open web steel joists. The intake structure houses and protects both safety-related and non-safety related systems and components.

Summary

To summarize, this LAR proposes to replace the working stress design method with the ultimate strength design method for "Normal Operating/Service Conditions (Operating Basis Earthquake OBE)". This change does not affect the two other conditions which include "No Loss of Function (Design Basis Earthquake DBE)" and "Accident Conditions (high differential pressure and temperature plus DBE)." This methodology change will be applied to all Class I structures except the containment structure, the SFP, and the foundation mats as discussed previously. This methodology is commonly used at other nuclear plants (See Section 4.2). Load combinations are developed based on applicable codes and equations for accident conditions currently defined in USAR Section 5.11.3.6.a.

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.

2.0.1 Concrete normal/service condition design (change from WSD to USD) – See Section 2.1 for Additional Information

The LAR proposes to replace the working stress design method with the ultimate strength design 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 proposed licensing basis (PLB) change is to normal service conditions where the USD method will replace the WSD method. 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	All Class I structures (except the containment structure, spent fuel pool (SFP), and the foundation mats).

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). 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 change for actual concrete strength described below is expected to eliminate the need to install a significant number of new columns. This minimizes the potential for harm to structures, safety-related equipment, or personnel caused by the installation of these columns inside containment where significant interferences exist.

2.0.2 Use of Actual Concrete Compressive Strength (Based on Original Test Data) – See Section 2.2 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.

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).
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 (i.e. 4000 psi for Class B concrete) with the actual compressive design strength based on original 28-day test data which meets the strength testing criteria within ACI 318-63 (i.e. not age hardening). Original test data shows that the actual compressive strength of the concrete exceeds the original design value (i.e. 4000 psi for Class B concrete). The method for determining the actual concrete compressive strength is discussed in detail within Section 2.2.
Where Applicable	As shown in Table 7 in Section 2.2.
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 2.2 below. 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 value to the actual concrete strength while meeting the testing requirements within ACI 318-63.
Consequences	See 2.0.1 above.

2.0.3 Use of Actual Reinforcing Steel Yield Strength (Use original test data for design basis) – See Section 2.3 for Additional Information

The proposal for reinforcing steel yield stress is to use a statistical increase from 40 ksi to 44 ksi. The use of test data for steel was included in the operability calculations and reviewed during the 2012 inspections.

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 (44 ksi) derived from plant construction data.
Where Applicable	CIS (RC&C, columns, beams, and slabs).
Margin	Adequate design margin is maintained because original test data shows that the mean yield strength with 95% confidence level supports the increased in yield strength to 44 ksi.

Consequences	This PLB change in combination with the changes discussed above (i.e. WSD to USD, actual concrete strength etc.) minimizes the need for physical modifications to CIS and is anticipated to eliminate the need for a significant number of columns. This minimizes the potential for harm to structures, safety-related equipment, or personnel caused by the installation of these columns inside containment where significant interferences exist. ACI 318-63, the current COR, uses the inverse of a phi factor of 0.9 and 0.85 applied to the load. This number is used to account for design inaccuracies, future changes, reliability of material strengths, dimension variations, etc., (Reference 6.5 Commentary Section 1504). The design is accurate and all changes are evaluated. There is a high level of confidence in the material strengths. The PLB change will continue to use the phi factor as prescribed by the COR.

2.0.4 Use of Dynamic Increase Factors (DIF) and Limit Design for RC&C – See Section 2.4 for Additional Information

The LAR proposes to allow use of dynamic increase factors (DIF) and limit design method for analysis of the RC&C.

Table 4 – DIF and Limit Design Method Summary

Current Licensing Basis	ACI 318-63 Code does not provide Limit Design Methodology. The CLB is silent on the use of DIFs.

Proposed Licensing Basis	The PLB adopts the application of DIFs when appropriate for the RC&C and limit design method. The DIFs will be utilized as specified in ACI 349-97, C.2.1 and will not be used if dynamic load factors (DLF) are less than 1.2. ACI 349-97, Appendix C is a commonly accepted design code used in the nuclear industry. The application of limit design method for RC&C walls includes deformation limit equations in ACI 349-97 Commentary on Appendix C Section C.3 The proposed application is similar to the guidance in Bechtel Power Corporation, Topical Report BC-TOP-9A, "Design of Structures for Missile Impact," Revision 2, approved for use in the evaluation of tornado missile impacts at FCS by Amendment No. 272. The Bechtel Report contains methods, including dynamic increase factors that evaluate how missiles impact structural elements. Although OPPD proposes a slightly different application, the use of DIF for missile dynamic events has been approved at FCS. However, instead of missile impacts, the RC&C will be evaluated using DIF for compartment pressurization.
Where Applicable	RC&C portion of the CIS.
Margin	Adequate design margin is maintained in using the limit design method and DIFs by the conservatism in the load combination equations for short-term high differential pressure.
Consequences	The limited clearances available in the RC&C make the placement of additional concrete structures in that area extremely difficult. Use of the proposed changes significantly reduces or eliminates the need for modifications in the RC&C. The safety benefit that would be gained by the installation of additional concrete structures in this area is small and has the potential to damage the nuclear steam supply system (NSSS) or harm workers if an accident were to occur during installation.

2.0.5 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 5 – 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 clarifications 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 CIS or the auxiliary building.

In summary, the changes described in sections 2.0.1 through 2.0.5 have several parts that along with anticipated physical modifications to a limited number of structures will be essential to the restoration of the design basis for CIS. The changes are not universally applied; application is limited to the specific areas described in this LAR.

2.1 Replace WSD with USD for Normal Operating/Service Conditions

For normal operating/service conditions, the WSD load combinations, ACI 318-63 design methods, and allowable stresses are replaced with equivalent USD factored load combinations and ACI 318-63 USD methods. 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) are used in the required ultimate strength capacity (U) for use in normal operating/service conditions remain consistent with those already accepted for use in the 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 6 – Load Combinations

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

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 6.11) in accordance with the guidance of Section 3.A. of Standard Review Plan (SRP) 3.8.4, Revision 4 (Reference 6.12). Note that these factors are less severe than those required by the ACI 318-63 Code for the ultimate strength design load combination of Section 1506 equation (15-1). However, the load factors applied to normal dead and live loads (1.5D +1.8L) in the ACI 318-63 Code have not been commonly adopted by commercial nuclear plants in the United States (See Section 4.2). The 1.4D and 1.7L load factors were implemented in the ACI 318-71 Code (References 6.13 and 6.17) 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. $1/\Phi$ (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 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 in the auxiliary building, intake structure, and containment internal structures. As noted previously, these changes will not be applied to the containment structure, the spent fuel pool, or the foundation mats.

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

2.2 Determination of Actual Concrete Compressive Strength Based on Original Test Data

The containment structure was originally designed using Class A concrete (specified as compressive strength 5000 psi). Other Class I structures (i.e. the auxiliary building, intake structure, and non-shell portions of the containment building) were originally designed using Class B concrete (specified as compressive strength 4000 psi).

As detailed below, the proposed change will allow reinforced concrete structures located in specified areas of the auxiliary and containment buildings to be evaluated using concrete compressive strength based on actual 28-day test data while meeting the strength testing requirements of the FCS licensing basis concrete Building Code ACI 318-63 Section 504 in lieu of the original specified value (i.e. 4000 psi for Class B concrete). This is appropriate as original test data shows that the actual 28-day compressive strength of the concrete exceeds the original specified value.

No change is proposed to the design criterion for compressive strength of concrete in the containment structure, intake structure, or the foundation mats.

Proposed Methodology for PLB Concrete Strength Determination

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 structures as shown in Table 7. Specifically, one concrete strength will be established for the auxiliary building and one concrete strength will be established for containment internal structures.

- Documentation of the new design basis concrete strengths will be recorded within FCS design basis calculations.
- The 28-day test data used will be limited to the areas to which the data applies, i.e. the auxiliary building and containment internal structures.
- 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 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 for each specific structure (additional conservatism beyond ACI 318-63 requirements).

<u>Justification for Proposed Methodology</u>

The actual concrete strengths determined with the methodology prescribed in this section satisfies 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.

The actual concrete strength may only be used in areas not potentially affected by degradation of concrete due to long-term exposure to high radiation, excessive moisture or harsh chemicals for prolonged periods. Areas of Class I structures where actual concrete strength may and may not be used are shown in Table 7.

Table 7 – Concrete Strength Applicable Areas

Area	Allowed Application of Original Test Data	Non-Allowed Application of Original Test Data[1]
Auxiliary Building	All areas except where not allowed.	Exterior walls at elevations below 1007 feet shall continue to use f_c' = 4000 psi.
Containment Internal Structures (CIS)	All areas except where not allowed.	Reactor cavity floor and concrete around the reactor vessel shall continue to use f_c' = 4000 psi.
Intake Structure and Foundation Mat	Not allowed.	Not applicable, will continue to use f_c' = 4000 psi.
Containment Structure / Foundation Mats	Not allowed.	Not applicable, will continue to use f_c' = 5000 psi.
Spent Fuel Pool	Not allowed.	Not applicable, will continue to use f_c' = 4000 psi.

Note: The application of actual concrete compressive strength in lieu of original specified design values is not allowed where structures undergo prolonged exposure to high radiation, excessive moisture, or harsh chemicals.

The containment structure, the spent fuel pool, the intake structure, and the foundation mats will continue to use the original specified design values for concrete compressive strength.

Aging management of these structures is described in Section 2.7 of this document.

2.3 Determination of Actual Reinforcing Steel Yield Strength Based on Original Test Data

Higher steel yield strength values are proposed for reinforcement of the CIS beams, slabs, and columns. Higher yield strength values are also proposed for reinforcing steel used in the re-evaluation of the walls of the reactor cavity and compartments (RC&C). The increase is limited to 44 ksi. Section 3.3 of this document provides technical justification for the use of the actual yield strength.

2.4 Use of Dynamic Increase Factors (DIF) and Limit Design for RC&C

The LAR proposes to allow use of dynamic increase factors (DIF) and limit design method for analysis of the RC&C. This change applies to the reactor cavity and compartment walls and transfer canal and is not being applied to other Class I structures.

In accordance with ACI 349-97, Appendix C, dynamic strength increase factors appropriate for the strain rates involved may be applied to static material strengths of steel and concrete for purposes of determining section strength. The dynamic strength increase factors are specified in Section C.2.1 of ACI 349-97. This change will allow use of DIFs for no loss-of-function conditions associated with compartment pressurization.

A gap analysis compared the ACI 349-97 Code to the COR. See Section 2.9 for results.

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 ultimate strength design 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.

2.6 Conditions that the Proposed Amendment will Resolve

2.6.1 Replace WSD with USD for Normal Operating/Service Conditions

For normal operating/service load cases, Fort Calhoun Station'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. For example, the USD method does not show flexural overstress in normal loading conditions as found during the re-constitution efforts for the CIS and auxiliary building using the WSD method. 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 on containment internal structures 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. Projects requiring new designs or re-evaluations of existing reinforced concrete structures can be conducted more efficiently with less chance for error as design calculations will only require USD member capacities. (This does not apply to the containment structure, which will continue to use the WSD method.) The current method requiring both WSD and USD calculations, is unnecessarily complex, and results in over-designed and over-engineered concrete structures. Furthermore, universities no longer teach working stress design and there are a diminishing number of engineers with the necessary experience, due partly to the fact that local government jurisdictions no longer accept this obsolete method.

The proposed load factors use a select load combination (1.4D + 1.7L) for the ultimate strength method to evaluate normal operating/service conditions and are consistent with 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. Although the ACI 349-97 load factors are lower than those required by the ACI 318-63 Code for the ultimate strength design load combination of Section 1506, Equation 15-1, U.S. commercial nuclear plants do not commonly use the ACI 318-63 Code load combination (i.e. 1.5D +1.8L). The proposed load factors better reflect ultimate strength design behavior.

Load combinations are developed for normal operating/service conditions using the no loss-of-function ultimate strength design 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: U = 0.75(1.4D + 1.7L + 1.7W) for use with the ultimate strength design method.

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

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

The use of the actual concrete compressive strength based on 28-day test data in lieu of the original specified 4000 psi in design re-evaluations will resolve conditions found during the re-constitution efforts for the CIS. 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 Actual Reinforcing Steel Yield Strength Based on Original Test Data

The use of actual reinforcing steel yield strength helps resolve bending overstresses in beams. The use of 10% higher yield strength for reinforcing steel is limited to approximately 1.65 standard deviations of the Certified Material Test Reports (CMTRs), which shows a high confidence level. The higher yield strength is restricted to the RC&C and to the CIS. 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 Limit Design Method and Use of Dynamic Increase Factors

Use of the limit design method and DIFs helps resolve overstress issues with the RC&C. Guidance for application of Dynamic Increase Factors (DIF) is included in ACI 349-97 Appendix C and associated commentary. Regulatory Guide 1.142 provides further guidance and identifies restrictions. Specifically, DIF can be realized only when material is subjected to high strain rates.

2.6.5 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 6.6 and 6.7 respectively), 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 6.10) 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."

FCS currently inspects the auxiliary building and containment internal structure per procedures SE-PM-AE-1001, Auxiliary Building Structural Inspection and SE-PM-AE-1004, Containment Building Structural Inspection. Each procedure has an inspection frequency of 3 years with caveats to increase or decrease the frequency as accumulated inspection findings warrant, but shall not exceed 5 years. A review of the results from recent inspections did not identify any significant structural deterioration that would invalidate the use of the CLB or PLB items requested by this LAR; the results demonstrate satisfactory health of the structures in question.

2.8 Analysis Supporting the License Amendment Changes

Detailed analysis is in progress to restore the CIS to design basis. The preliminary results of the beam evaluations based on CLB and PLB criteria suggest that a majority of the CIS can be restored on the basis requested in this LAR. Table 8 below shows the preliminary results from the CLB calculation prepared in 2012 compared to the analysis performed to date while implementing the changes to the license basis (PLB). Using the CLB, 19 of the 22 total beams require modification. Using the PLB preliminary analysis, 4 of the 22 total beams may require modification.

Table 8 – Summary of IRs for CLB and PLB Conditions

CIS Beams	Current License Basis (CLB)		Proposed License Basis (PLB)
	WSD IR	USD IR	USD IR with new load factors (preliminary)
Beams on Elevation 1013'			
B5-13	1.04	0.81	0.80-0.90
B13-13	1.27	0.86	0.70-0.80
B14-13	0.82	0.73	N/A
B43-13	1.30	1.00	0.70-0.80
B44-13	1.48	1.09	0.80-0.90
B21-13	1.46	1.44	1.20-1.30
B22a-13	1.65	2.47	1.20-1.30
B22b-13	1.66	2.44	1.20-1.30
B23-13	1.65	1.43	1.05-1.15
B36-13	1.30	2.17	0.85-0.95
B46b-13	1.05	1.19	0.80-0.90
B27-13	1.12	0.92	0.70-0.80
B29-13	2.45	2.38	0.80-0.90
Beams on Elevation 1045'			
B12-45	1.90	1.75	0.90-1.00
B21a-45	0.65	0.44	0.30-0.40

CIS Beams	Current License Basis (CLB)		Proposed License Basis (PLB)
	WSD IR	USD IR	USD IR with new load factors (preliminary)
B21b-45	0.65	0.44	0.30-0.40
B23-45	1.32	0.81	0.75-0.85
Beams on Elevation 1060'			
B56a-60	1.02	0.74	0.70-0.80
B61a-60	1.19	1.20	0.90-1.00
B61b-60	1.16	1.18	0.90-1.00
B64-60	0.88	1.11	0.60-0.70
B65-60	1.15	0.93	0.90-1.00

Notes

1. The IRs presented in this table do not include reductions that result from modifications.
2. The preliminary PLB analysis results (LAR methodology) presented in the table above include elimination of WSD, use of 44ksi reinforcing steel yield strength, use of 5440psi concrete strength, and new load combinations 1.4D+1.7L and 1.05D+1.25(L+E).
3. The IRs over 1.0 for beams B22a-13 and B22b-13 are being resolved in RFO28 to meet CLB conditions in accordance with the Regulatory Commitment made in OPPD letter LIC-15-0042. Remaining beams with PLB IRs greater than 1.0 will be resolved with future modifications depending on the results of the final analysis.

Detailed analysis results have not been completed for the auxiliary building, RC&C, or intake structure. However, preliminary analysis indicate PLB conditions will minimize the modifications required to restore these structures to the design basis.

2.9 Gap Analysis of ACI 318-63 to ACI 349-97

2.9.1 Scope

The scope of the gap analysis is to compare ACI 349-97 to the COR and evaluate the differences between the two codes. The comparison looks specifically at the margin associated with concrete design for applying DIFs and the limit design method to the RC&C.

ACI 349-97 is based on ACI 318-89 (Revised 1992) – with an exception that Chapter 12 is based on ACI 318-95.

2.9.2 Methodology

A section-by-section comparison of the language and material between the strength design provisions of ACI 349-97 and the original design basis working stress design method along with the ultimate strength design method of the ACI 318-63 was performed. Dissimilarities are summarized as follows:

- Design load combinations (ACI 349-97, Chapter 9)—ACI 349-97 aligns code provisions by introducing nuclear safety-related load combinations regarding normal, severe environmental, extreme environmental, and abnormal loads. However, the Fort Calhoun load combinations also apply ultimate strength load factors that are different but comparable to both Codes. In addition, the strength reduction factors listed in the USAR are consistent with both Codes.

Other differences were identified due to expanded or new provisions as follows:

- Slenderness effects (ACI 349-97, Chapter 10) ACI 349-97 adds to the provisions to account for member stiffness and moment magnification in computation of second-order demand load effects.
- Shear-friction considerations (ACI 349-97, Chapter 11) ACI 349-97 adds equations to introduce shear-friction reinforcement design provisions.
- Special seismic requirements for steel reinforcing detailing (ACI 349-97, Chapter 21) ACI 349-97 adds special detailing provisions for seismic that are not related to impactive or impulsive conditions.
- Special provisions for thermal loading (ACI 349-97, Appendix A) ACI 349-97 addresses the consideration of thermal stresses and thermal strains for both normal operating and thermal accident conditions.
- Design of steel embedment (ACI 349-97, Appendix B) ACI 349-97 provides minimum requirements for design and anchorage of steel embedments to bring

consistency between ACI and AISC practice and this is not related to impactive or impulsive conditions for the RC&C walls.

- Special provisions for impulsive and impactive loadings (ACI 349-97, Appendix C) provides design provisions to account for time-dependent loads due to collision of masses and those time-dependent loads not associated with collision of finite masses, including the concept of dynamic strength increase (DIF), which concerns the strain rates of the load effects.

Based on the review of the provisions of ACI 349-97 and an assessment of the dissimilarities to the ACI 318-63 Code, the provisions of the ACI 349-97, Appendix C are compatible with the ACI318-63 design methodology for Fort Calhoun Station with regard to Dynamic Increase Factors.

2.9.3 Findings

Most of the code sections that are presented in ACI 349-97 are similar to ACI 318-63 and do not adversely affect the assessment of the design basis compared to the original license basis. There are technical aspects of ACI 349-97 that influence the structural re-evaluation of the existing FCS concrete structures and have been reconciled.

2.9.4 Conclusions

Based on the comparison of ACI 349-97 Code and the dissimilarities to ACI 318-63 it has been determined that use of DIF as described in ACI 349-97 and Regulatory Guide 1.142 would be advantageous for re-evaluation of the RC&C. Use of the USD methodology is adequate to evaluate the RC&C.

2.10 Quality Program During Construction and Quality of Material Tests

The quality program (Reference 6.25) was implemented at the Fort Calhoun Station 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 6.26) 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 6.27 records a discussion confirming the requirements for controlling concrete. Reference 6.27 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.*

In conclusion, 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 license amendment request.

3.0 TECHNICAL EVALUATION

3.1 Use Ultimate Strength 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 6.16). The materials, construction, and detailing requirements are unchanged. The ACI 318-63 Code included both working stress design and ultimate strength design methods just before the industry began phasing out the application of working stress design for reinforced concrete in the 1960s. In 1971, the American Concrete Institute labeled working stress design 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. 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 since USD design methodologies require factored loads in combinations that represent the normal operating condition of the plant. The proposed load factors in the first revised load combination (1.4D + 1.7L) will be used for ultimate strength design 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). Commercial nuclear plants in the United States do not commonly use this load combination (i.e. 1.5D +1.8L) from the ACI 318-63 Code. The proposed load factors better reflect ultimate strength design behavior, see discussion in Section 9.3.1 of ACI 318R-71 (Reference 6.17).

Load combinations are developed for normal operating/service conditions using the no loss-of-function ultimate strength design method equation from USAR Section 5.11 (i.e. $1/\phi(1.0\ D \pm 0.05D + 1.25\ Pc + 1.25\ E + 1.0\ Tc)$) 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: $U = 0.75\ (1.4\ D + 1.7\ L + 1.7\ W)$ for use with the ultimate strength method. The factored load combination for normal load conditions using the ultimate strength method are a simple conversion to consider loads in combination with the hydrostatic load (F).

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

The request to implement actual 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 compressive strength for Class B concrete. The use of actual concrete compressive strength applies to concrete (not related to age hardening) 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 actual 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 actual concrete compressive strength above the minimum specified CLB COR 4000 psi value. 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 actual concrete strength will be established using analysis of original lab test data meeting the testing requirements of ACI 318-63. The data analysis method to be used is described in Section 2.2 of this document.

Application of the actual concrete strength will be limited to areas in the auxiliary building and containment internal structures that are not exposed to excessive moisture, high radiation, or harsh chemicals for prolonged periods.

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

Quality records show that there were 115 heat code samples used in the construction of the CIS. Some of the heat code yield stress values could not be identified and 105 of the 115 samples are known. Based on 105 samples specifically used for CIS, the 95% confidence level is equal to 44.45 ksi. As a result, the current design steel yield strength (i.e. 40 ksi) is increased to 44 ksi with high confidence for the RC&C and for CIS (Reference 6.14).

3.4 Use of Dynamic Increase Factors (DIF) and Limit Design for RC&C

The request to allow the inclusion of the Limit Design Method is required because there is no equivalent method in the ACI 318-63 Code. The original design of the reactor cavity and compartment walls (RC&C walls) was based on the limit state of the reinforced concrete structure. Design forces and moments at the limit state, which were used to determine sizes and placements of reinforcing bars (rebar), cannot be replicated by any form of linear finite element analysis, herein called the Linear Design Method. They can only be computed by using non-linear (or step-wise linear) finite element analysis, herein called the Limit Design Method. While the Limit Design Method is able to reasonably reproduce the original design forces and moments at the limit state by simulating load redistribution behavior, the Linear Design Method is not.

When the Limit Design Method is applied, where flexure controls design of the RC&C walls, rotations of the walls in any yield zone must be less than the rotational capacity of the zone, as expressed by Equations 3.4.1 and 3.4.2 in Section C.3.4 of ACI 349R-97 Appendix C (ACI 349-97 Commentary). It is important to note that the Limit Design Method applies to all load combinations applicable to the RC&C walls.

The request to allow the inclusion of dynamic increase factors (DIF) is required because there are no equivalent factors in the ACI 318-63 Code. Accidental pressure loads result from high-energy line breaks inside the RC&C walls. Because of their impulsive nature and rapid strain rates that occur in structural elements under them, both the concrete and reinforcing steel will exhibit strengths higher than those under static loading conditions. DIFs represent the ratio of dynamic to static strengths. OPPD requests that DIFs may be utilized as specified in Section C.2.1 of ACI 349-97 Appendix C. As restricted in Regulatory Guide 1.142 (Reference 6.23), DIFs will not be used if dynamic load factors (DLF) are less than 1.2. It is important to note that DIFs apply only to compartmental pressures.

The application of the limit design method and DIFs is similar to the guidance in Bechtel Power Corporation Topical Report BC-TOP-9A "Design of Structures for Missile Impact", Revision 2 (Reference 6.21), which was recently approved (Reference 6.22) for Fort Calhoun Station.

3.5 Clarification of Hydrostatic Load in the USAR Section 5.11

The controlled hydrostatic load is changed from live load to dead load for ultimate strength design 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.

4.0 REGULATORY EVALUATION

4.1 Applicable Regulatory Requirements/Criteria

Fort Calhoun Station 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 Fort Calhoun Station 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 are USAR Appendix G, Criterion 2, *Performance Standards* and USAR Appendix G, Criterion 10, which pertains to the containment building. Criteria 2 and 10 as described in USAR Appendix G are 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.

CRITERION 10 - CONTAINMENT

Containment shall be provided. The containment structure shall be designed to sustain the initial effects of gross equipment failures, such as a large coolant boundary break, without loss of required integrity and, together with other engineered safety features as may be necessary, to retain for as long as the situation requires the functional capability to protect the public.

This criterion is met. The containment structure is designed to sustain the initial effects of gross equipment failures, such as a large reactor coolant boundary break, without loss of required integrity and, together with other engineered safety features as necessary, to retain for as long as necessary, the functional capability to protect the public.

The containment building is designed to withstand an internal pressure of 60 psig at 305°F including all thermal loads resulting from the temperature associated with this pressure with a leakage rate of 0.2% or less of the contained volume per 24 hours and will be subject to a leak rate and pressure test to demonstrate compliance with the design.

The changes proposed by this LAR will continue to ensure that Criterion 2 and Criterion 10 are met.

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 (not age hardening) in lieu of the original specified value of 4,000 psi for Class B concrete. Application of the actual compressive strength will be limited to areas in the auxiliary building and containment internal structures where the concrete is not exposed to excessive moisture, high radiation, or harsh chemicals for prolonged periods. The 10% increase in yield strength is supported by statistical analysis of the original test data.

Inclusion of the Limit Design Method and dynamic strength increase factors (DIF) fills a gap in the ACI 318-63 Code. The proposed application is similar to the guidance in Bechtel Power Corporation, Topical Report BC-TOP-9A, "Design of Structures for Missile Impact," Revision 2, approved for use in the evaluation of tornado missile impacts by Amendment No. 272. The DIFs will be utilized as specified in ACI 349 C.2.1 and will not be used if dynamic load factors (DLF) are less than 1.2. ACI 349-97, Appendix C is an accepted design code used in the Nuclear Industry.

4.2 Precedent

The ACI 318-63 Code was used for the design of nuclear power plants constructed in the era that Fort Calhoun Station was built. Operating nuclear power plants licensed to use the ACI 318-63 ultimate strength design 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, Section 3.8.5.4.2 of 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.

The use of dynamic strength increase factors (DIF) are specified in Section C.2.1 of ACI 349-97 and is a method previously endorsed by the NRC in SRP 3.8.4 and RG 1.142.

The ACI 318-63 Code is the current licensing basis code of record at FCS. Use of actual concrete compressive strength applies to concrete meeting the strength testing requirements of Section 504 in ACI 318-63.

4.3 No Significant Hazards Consideration

This License Amendment Request (LAR) proposes to revise Fort Calhoun Station (FCS) Unit No. 1, Updated Safety Analysis Report (USAR), Section 5.2 "Structure Materials of Construction," and 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 Class I structures at FCS.

The Omaha Public Power District (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 design new or re-evaluate existing Class I structures other than the containment structure (cylinder, dome, and base mat), the spent fuel pool (SFP), and the foundation mats. These structures will continue to utilize the current license basis and thus are not affected by this change. The proposed change allows other Class I structures 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 Class I structures meet industry accepted ACI Code requirements. This LAR does not propose changes to the no loss-of-function loads, loading combinations, or required ultimate strength capacity.

Calculations that apply the limit design method and use dynamic increase factors (DIF) of ACI 349-97, Appendix C will demonstrate that the concrete structures meet required design criteria. Therefore, these proposed changes will not pose a significant increase in the probability or consequences of an accident previously evaluated.

The use of actual concrete strength based on original test data for the areas identified in Section 2.2 of this document and the use of 10% higher steel yield strength for the reactor cavity and compartment (RC&C) and containment internal structures (CIS) 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 ultimate strength design 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 (SSC). 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 Class I structures will continue to ensure that those structures 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. Calculations that apply the limit design method and use dynamic increase factors (DIF) of ACI 349-97, Appendix C will demonstrate that the concrete structures meet required design criteria. Use of the actual 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 10% higher steel yield has minimal reduction on design margin for the RC&C or the CIS. The controlled hydrostatic load is changed from live load to dead load for ultimate strength design in the definition which is consistent with ACI-349-97.

The use of these alternative methodologies for qualifying Class I structures 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 or re-analysis of existing Class I structures with the exception of the containment structure, the spent fuel pool, and the foundation mats for which no change to the current licensing basis (CLB) is proposed.

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

Use of the actual compressive strength of concrete based on 28-day test data (not age hardening) is justified and further constrained by limiting its application to areas where the concrete is not exposed to harsh conditions. ACI 349-97, Appendix C is an accepted design code used in the nuclear industry. Calculations using DIFs per ACI 349-97, Appendix C must demonstrate that the Class I structures continue to meet an appropriate design code widely used in the nuclear industry. The use of a 10% higher steel yield was conservatively derived from original test data and has minimal reduction on design margin for the RC&C or the CIS. The controlled hydrostatic load is changed from live load to dead load for ultimate strength design 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

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

5.0 ENVIRONMENTAL 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.

6.0 REFERENCES

6.1 Fort Calhoun Station Unit 1 Updated Safety Analysis Report (USAR) Section 5.11, "Structures Other Than Containment"

6.2 Fort Calhoun Station Unit 1 Updated Safety Analysis Report (USAR) Section 5.2 "Materials of Construction"

6.3 Fort Calhoun Station Unit 1 Updated Safety Analysis Report (USAR) Section 5.5, "Containment Design Criteria"

6.4 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)

6.5 ACI 318-63, Building Code Requirements for Reinforced Concrete, American Concrete Institute

6.6 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)

6.7 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)

6.8 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)

6.9 Letter from NRC (M. Hay) to OPPD (L. P. Cortopassi), "Fort Calhoun – NRC Integrated Inspection Report Number 05000285/2014007," dated May 14, 2014 (NRC-14-0053) (ML14134A410)

6.10 Program Basis Document (PBD)-42, "Structures Monitoring," Revision 4

6.11 ACI 349-97, Code Requirements for Nuclear Safety Related Concrete Structures, American Concrete Institute

6.12 NUREG-0800, Standard Review Plan, Section 3.8.4, "Other Seismic Category I Structures," Rev. 4, 09/2013

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

6.14 DIT-SA-13-005, "Use of Higher Reinforcement Yield Strength for Operability Calculations"

6.15 ACI 214-65, Recommended Practice for Evaluation of Compression Test Results of Field Concrete, American Concrete Institute.

6.16 ACI 318-63, Publication SP-10, Commentary on Building Code Requirements for Reinforced Concrete, American Concrete Institute

6.17 ACI 318R-71, Commentary on Building Code Requirements for Reinforced Concrete, American Concrete Institute

6.18 Not Used.

6.19 Not Used.

6.20 Not Used.

6.21 Not Used.

6.22 Regulatory Guide 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," Rev. 2, November 2001

6.23 Not Used.

6.24 Quality Assurance Program (WIP 008372).

6.25 Quality Assurance Records Transmittal For Batch Documents Number 83-268 (WIP 5932), Dated June 17, 1983

6.26 Letter from J. Woolsey to OPPD (J. Gassman, C. Murphy, F. Wittlinger, C. Mann) "Concrete Control," dated February 5, 1969 (WIP 013439)

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

Mark-up of Updated Safety Analysis Report

Section 5.2
Section 5.11

USAR-5.2

Structures

Materials of Construction

Rev 8

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5.2 Materials of Construction

The basic specifications for the primary structural elements of containment construction are presented in Sections 5.2.1 through 5.2.4. During the Nuclear Steam Supply System Refurbishment Project (NSSSRP) in 2006, a temporary opening was provided in the cylindrical wall for the entry of major equipment components into the containment. The NSSSRP construction opening is described in Section 5.3.6.

5.2.1 Concrete

Concrete for the NSSSRP construction opening is described in Section 5.3.6.

Ingredients

Cement conformed to ASTM C150 Type II with low alkali content. The maximum alkali content calculated as sodium oxide was limited to 0.50 percent.

The air entraining agent used was "MBVR" (Master Builders Vinsol Resin). This material conformed to ASTM C260.

The water reducing agent used was "Pozzolith Low Heat" as manufactured by the Master Builders Co. This material conformed to ASTM C494 Type A.

Fine aggregate is Platte River Valley sand gravel with 1/4 inch maximum size.

Coarse aggregate consists of 1-1/2 inch crushed limestone (3/4 inch for special limited applications) from the Plattsmouth ledge at Weeping Water, Nebraska.

The aggregate materials conformed substantially to ASTM C33, except for minor variations in gradation. The aggregate mix was based on the requirements of the Nebraska Standard Specifications for Highway Construction, modified slightly in accordance with local practice.

Investigation Program

The aggregates for use in the concrete mix were tested for acceptability. The tests were performed by the Omaha Testing Laboratory of Omaha, Nebraska, in accordance with the then current editions of the following ASTM standards:

a. ASTM C40, Organic Impurities in Sands for Concrete;

b. ASTM C75, Sampling Stone, Slag, Gravel, Sand and Stone Block for Use as Highway Materials;

c. ASTM C87, Effect of Organic Impurities in Fine Aggregate on Strength of Mortar;

d. ASTM C88, Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate;

e. ASTM C117, Materials Finer than No. 200 Sieve in Mineral Aggregate by Washing;

f. ASTM C123, Lightweight Pieces in Aggregate;

g. ASTM C127, Specific Gravity and Absorption of Coarse Aggregate;

h. ASTM C128, Specific Gravity and Absorption of Fine Aggregate;

i. ASTM C131, Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine;

j. ASTM C136, Sieve or Screen Analysis of Fine and Coarse Aggregates;

k. ASTM C142, Clay Lumps in Natural Aggregates;

l. ASTM C235, Scratch Hardness of Coarse Aggregate Particles;

m. ASTM C289, Potential Reactivity of Aggregates;

n. ASTM C295, Petrographic Examination of Aggregates for Concrete;

o. ASTM C324, Potential Volume Change of Cement Aggregate Combinations.

Mixes

The design and associated tests of concrete mixes was also performed by the Omaha Testing Laboratory. Concrete strengths were predicated on the basis of ACI-301, "Specification for Structural Concrete for Buildings", Section 308, Method 2 for Ultimate Strength Type Concrete and Prestressed Concrete.

The basic concrete mixes evolved and utilized for construction are shown in Table 5.2-2.

Table 5.2-2 - Concrete Mix Compositions

Application	Containment Mat & Shell	Special Limited Applications*
Aggregate Size, in	1-1/2	3/4
Compressive Strength, psi at 28 days	5000	5000
Water-Cement Ratio, gals, max/sack	4.25	4.00
Cement Content, max sacks/ cu. yd.	6.25	7.00
Aggregate Proportions, %		
Sand Gravel	50 ± 3	50 ± 3
Crushed Limestone	50 ± 3	50 ± 3
Air Content, %	4.75 ± 0.75	4.75 ± 0.75
Pozzolith Low Heat, lbs/ sack cement	0.2	0.2
Slumps, in, maximum	3	3
Density, lb/cu. ft, minimum	145	145

*This mix was a special mix utilizing 3/4 in. maximum crushed limestone for special limited applications where the density of reinforcement and difficulty of ensuring good placement of 1-1/2 in. aggregate concrete warranted its use. Its application has been restricted to the portion of the structure in the immediate vicinity of the anchorages for the helical tendons.

5.2.2 Reinforcing Steel and Cadweld Splices

Reinforcing Steel and Cadweld Splices for the NSSSRP construction opening are described in Section 5.3.6.3.

Reinforcing steel in the foundation mat consists of deformed billet steel bars which conformed to ASTM A615, Grade 60. This steel has a minimum yield strength of 60,000 psi, minimum tensile strength of 90,000 psi, and a minimum elongation of 7 percent in an 8 inch specimen.

Reinforcing steel in the cylindrical wall, the domed roof and around openings is deformed billet steel which conformed to ASTM A615, Grade 40. This steel has a minimum yield strength of 40,000 psi, minimum tensile strength of 70,000 psi, and a minimum elongation of 7 percent in a 8 inch specimen.

Splices for reinforcing steel conformed to the requirements of ACI Code 318-63. Splices for bar sizes No. 11 and smaller were lapped except at certain locations such as at the construction openings. Splicing for bars at these locations and for all bar sizes larger than No. 11 was accomplished by approved positive connectors, specifically Cadweld "T" series connectors, (as manufactured by Erico Products Incorporated of Cleveland, Ohio), designed to develop the specified tensile strength of the reinforcing steel.

No reinforcing steel strength or tack welding was permitted.

5.2.2.1 Reactor Cavity and Compartments and Maintenance Activities

For the reactor cavity and compartments, the yield strength (F_y) of reinforcing steel is based on testing in accordance with ASTM Standards. Historical laboratory test data of Grade 40 steel reinforcement used in the construction of Fort Calhoun Station Class I structures indicate that the yield strength test results exceed the material specification values used in the original design. Statistical analysis of Certified Material Test Reports (CMTR) for the CIS confirms the 95% confidence yield strength for Grade 40 reinforcing steel to be 44 ksi. This is applicable to the reactor cavity and compartments and raises the yield strength from 40 ksi to 44 ksi.

The higher reinforcing steel strength is also used to evaluate the containment internal structure beams, slabs, and columns.

5.2.3 Prestressing Post-Tensioning System

Section 5.3.6.4 describes the prestressing system and the NSSSRP construction opening.

5.2.3.1 Description

The prestressing system is the 90 wire BBRV system as furnished by the Inland-Ryerson Construction Products Company.

Each tendon consists of 90 parallel 1/4 inch diameter high tensile cold drawn, stress relieved wires. Each end of the tendon terminates in an anchorage assembly which transfers the tendon force to the concrete structure. Positive anchorage of the wire ends is achieved with 3/8 inch diameter button heads, cold formed at the wire ends.

The anchorage system consists of:

a) A stressing head on one end of the tendon and a stressing head/shim nut combination on the other end of the tendon: The cold formed button head on the tendon wire end transmits the tendon force to the stressing head. The stressing head or stressing head/shim nut is engaged through the external buttress threads by a hydraulic stressing device which induces the desired force in the tendon.

b) A set of split tube shims: These shims are installed between the stressing head or stressing head/shim nut and the bearing plate; they maintain the strain induced in the tendon and transmit the tendon force from the stressing head to the bearing plate.

c) A bearing plate to transfer the tendon force to the concrete surface.

The tendons were installed in the concrete structure in steel conduits, rigid or semi-rigid depending on location, and fully interlocked and completely sealed from terminal to terminal against leakage of mortar into the conduit.

5.2.3.2 Material Properties

Tendon wire was cold drawn and stress relieved, with a minimum ultimate strength of 240,000 psi. Each 90 wire tendon has a minimum ultimate strength of 1,060,000 pounds and a minimum yield strength of 848,000 pounds.

Anchor components were designed to work within their elastic range up to a load equal to the ultimate strength of the tendon, except for the plastic seating of the button heads.

Governing prestressing material specifications were as follows:

a) Tendon wires: ASTM A421, Type BA;

b) Bearing plate: ASTM A36, (Fine Grained Practice);

c) Anchor head: AISI 1141 Special Quality;

d) Bushing: HFSM Tubing AISI 1045 or AISI 4140;

e) Shims: HFSM Tubing AISI 1026;

f) Tendon sheathing:

1. Rigid: 4 inch O.D., 14 gage and 3 gage flash controlled welded tubing;

2. Semi-rigid: 3-3/4 inch I.D. 22 gage spirally formed from steel strip with interlocked seam.

5.2.3.3 Corrosion Protection

Tendons were prepared at the factory for shipping by application of a coating of a thin-filmed temporary corrosion inhibitor. Each tendon was then coiled and enclosed in a plastic bag, and 4 ounces of Shell No. 250 VPI (vapor phase inhibitor) powder was dusted into each bag. Each bag was then sealed and shipped to the job site.

The temporary corrosion inhibitor contained not more than 10 ppm each of chlorides, nitrates and sulfides. It was designed to adhere to the tendon during installation in the structure and to provide corrosion protection under the prevalent field conditions.

The permanent corrosion protection system subsequently applied to the tendons was manufactured and supplied by the same manufacturer and has the same chemical restrictions as the temporary corrosion inhibitor to ensure that all materials used in the compounding of both systems were compatible.

The exposed portion of the bearing plates, which are located in the stressing galleries and receive the tendon stress, were coated with grease after final tendon installation to prevent rusting.

5.2.4 Liner Plate and Penetrations

Liner plate for the NSSSRP construction opening is described in Section 5.3.6.2.

The liner plate is 1/4 inch in thickness. The materials specified for the various components of the containment liner are in accordance with the following:

a. Liner plate at the foundation mat and within the recess below the reactor conformed to ASTM A516, Grade 60, except that Charpy vee-notch impact tests in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Class B vessels, were performed at a maximum temperature of 0°F.

b. Material for the cylinder and dome liner plate, for embedded structural shapes and stiffeners, for test channels, for reinforcing plates around penetrations, for personnel air lock internals and for the polar crane, with exception of the thickened liner plate, conformed to ASTM A36.

c. The thickened liner plate at the crane supports conformed to ASTM A516, Grade 60. In addition these plates meet and in special areas exceeded the requirements of ASTM A435 for ultrasonic testing.

d. Plate for the personnel air lock barrel, bulkhead and doors, and for the equipment hatch barrel and cover and surrounding reinforcing plates, conformed to ASTM A516, Grade 60, manufactured to ASTM A300 requirements, except that Charpy vee-notch impact tests were in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Class B vessels, and were performed at a maximum temperature of 0°F.

e. Material for penetration sleeves conformed to ASTM A333, Grade 1, for seamless pipe and ASTM A516, Grade 60, manufactured to ASTM A300 requirements for pipe formed from rolled plate, except that Charpy vee-notch impact tests were in accordance with the ASME Boiler and Pressure Vessel Code Section III, Class B vessels, and were performed at a maximum temperature of 0°F.

f. Bolting materials for the equipment hatch cover conformed to ASTM A320.

g. Material for forged flanges conformed to ASTM A350 except for impact test requirements; material for flanges fabricated from plate conformed to ASTM A516, Grade 60, manufactured to ASTM A300 requirements. Charpy vee-notch impact tests were in accordance with the ASME Boiler and Pressure Vessel Code Section III, Class B vessels, and were performed at a maximum temperature of 0°F.

h. Pipe for pressure connections to the test channels over liner seam welds inaccessible for inspection conformed to ASTM A106, Grade B.

Materials for penetrations and openings which must resist full design pressure conformed to the requirements of the ASME Boiler and Pressure Vessel Code, Section III, Class B vessels. NDT considerations were accounted for in paragraph N-1211a, ASME Boiler and Pressure Vessel Code, Section III, which required that Charpy V-specimens be tested at a temperature at least 30°F lower than the lowest service metal temperature in accordance with the requirements of paragraphs N-331, N-332, N-511.2 and N-515. The lowest service metal temperature for penetrations within the containment is 50°F. All those penetrations projecting outside the containment are located within the enclosure provided by the auxiliary building; the lowest service metal temperature for these penetrations is 50°F.

5.2.5 Protective Coatings and Paints Inside the Containment

Protective coatings for the NSSSRP construction opening are described in Section 5.3.6.2.

Construction

Coatings used within the containment were specified to withstand accident conditions. The applicable requirements of ANSI N101.4-1972, Quality Assurance for Protective Coatings Applied to Nuclear Facilities, and Regulatory Guide (RG) 1.54, Revision 0, Quality Assurance Requirements for Protective Coatings Applied to Water Cooled Nuclear Power Plants, were implemented for modification activities and meet or exceed original plant specifications and manufacturer recommendations.

Most carbon steel original equipment has prime coats of Carbozinc or Carboweld and finish coats of Phenoline 305, all products of the Carboline Corporation. The Safety Injection Tanks were primed with DuPont Corlar 825-8031 epoxy zinc chromate primer number 583 while the Pressurizer Quench Tank was primed with DuPont number 773 Dulux zinc chromate primer. Documentation that all primers and top coats stated above have been tested or are acceptable for their service applications is shown by Carboline Corporation and DuPont specifications and test sheets.

Concrete inside the containment was treated with Amercoat Nu-Klad 1100A and top coated with Amercoat No. 66. Testing has proven this type of coating to be adequate under accident conditions.

Some minor components such as instrument housings and electrical cabinets have proprietary coatings, mostly baked enamels. The total area so coated, however, is very small compared to the total surface area.

Maintenance and Modification

Coating materials currently used for maintenance of existing surfaces or for surfaces of new modifications installed inside the reactor containment are epoxy materials tested to withstand Fort Calhoun design basis accident (i.e., loss of coolant accident (LOCA)) conditions. The coating materials and their application comply with the requirements of RG 1.54. Each coating was tested in accordance with ANSI N101.2, Protective Coatings (Paints) for Light Water Nuclear Reactor Containment Facilities, or ASTM D3911, Evaluating Coatings Used in Light-Water Nuclear Power Plants at Simulated Design Basis Accident (DBA) Conditions, to conditions that bound the Fort Calhoun DBA conditions. Prior to exposure to the simulated DBA conditions, each coating test panel was irradiated to an accumulated dose of at least 1.8×10^8 Rads in accordance with ASTM D4082, Standard Test Method for Effects of Gamma Radiation on Coatings for Use in Light-Water Nuclear Power Plants.

Use of the above coatings precludes the possibility of large flaking or skinning off of sheets of paint during accident conditions. Since the only way use of these coatings could affect performance of engineered safety features is by flaking off and entry through the drains during recirculation after an accident, it is expected that their use will have no deleterious effects on engineered safety equipment and that flow blockage or fouling of heat transfer surfaces will not take place.

5.2.6 Compressive Test Strength of Concrete

Specified compressive concrete strength (f_c') is used for design of Class I structures except for the structures defined below where actual compressive strength has been established.

Where applicable, actual concrete design compressive strength is established using analysis of 28-day test data based on Building Code ACI 318-63 Section 504(c).

For structures designed in accordance with Part IV-A of ACI 318-63, 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 structures designed in accordance with Part IV-B of ACI 318-63, 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

A rolling average is calculated for consecutive strength tests, and the minimum strength of the average values represents the area specified. The design strength is the lesser of the minimum rolling average design basis concrete strength or the 95% confidence level of all test data for the structural group.

Concrete strengths based on 28-day test data may only be used in areas not potentially affected by degradation of concrete by long-term exposure to high radiation, damaging chemicals, or excessive moisture.

Actual concrete design compressive strengths are established using the methodology described above in the following areas:

- Auxiliary Building: Except for exterior walls below elevation 1007 ft.

- Containment Internal Structures: Except for the concrete walls surrounding the reactor vessel and the floor of the reactor cavity.

Actual concrete design compressive strength is not used for the Intake Structure or its foundation, the Spent Fuel Pool, Containment Structure or the Containment / Auxiliary Building foundation.

USAR-5.11

Structures

Structures Other Than Containment

Rev 14

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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
~~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~~

The allowable stress capacity of Class I structural steel is determined in accordance with the allowable stress design provisions from the AISC Code, 1963 edition.

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 + L~~

~~S = D + L + W or E~~

~~S = D + 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.~~

Class I concrete structures were originally designed for operating basis conditions using ACI 318-63 Code working stress capacities as discussed below in Section 5.11.3.4(b)

a. Ultimate Strength Design

With the approval of Amendment No. XXX, the design criteria for operating basis conditions changed to implement the ultimate strength design method for normal/operating service conditions for changes and reanalysis using the following load combinations:

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

$$U = 1\ (1.0D \pm 0.05\ D + 1.25L + 1.25W + 1.25H)$$

$$\Phi$$

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

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

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.

b. Working Stress Design

Foundations for Class I structures are not upgraded to ultimate strength design by Amendment XXX. They are designed on the basis of working stress for the following load combinations:

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

where:

S = Required section capacity

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

The application of limit design method for RC&C walls includes deformation limit equations in ACI 349-97 Commentary on Appendix C Section C.3.

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.

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

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

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

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

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~~

The application of limit design method for RC&C walls includes deformation limit equations in ACI 349-97 Commentary on Appendix C Section C.3

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. Dynamic increase factors appropriate for the strain rates involved for impulsive loads to the reactor cavity and compartment walls and transfer canal comply with ACI 349-97, Appendix C. 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 Sections 5.11.3.1a and 5.11.3.6.

5.11.3.8 Concrete Compressive Strength

Where applicable, concrete design compressive strengths (f_c`) may be established using analysis of historical test data as described in Section 5.2.6.

5.11.3.9 Steel Reinforcing Capacity

Where applicable, steel reinforcing capacity allowable stress values may be established using analysis of historical test data as described in Section 5.2.2.1.

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 5.2
Section 5.11

USAR-5.2

Structures

Materials of Construction

Rev ~~8~~

Safety Classification:

Safety

Usage Level:

Information

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Fort Calhoun Station

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5.2 Materials of Construction

The basic specifications for the primary structural elements of containment construction are presented in Sections 5.2.1 through 5.2.4. During the Nuclear Steam Supply System Refurbishment Project (NSSSRP) in 2006, a temporary opening was provided in the cylindrical wall for the entry of major equipment components into the containment. The NSSSRP construction opening is described in Section 5.3.6.

5.2.1 Concrete

Concrete for the NSSSRP construction opening is described in Section 5.3.6.

Ingredients

Cement conformed to ASTM C150 Type II with low alkali content. The maximum alkali content calculated as sodium oxide was limited to 0.50 percent.

The air entraining agent used was "MBVR" (Master Builders Vinsol Resin). This material conformed to ASTM C260.

The water reducing agent used was "Pozzolith Low Heat" as manufactured by the Master Builders Co. This material conformed to ASTM C494 Type A.

Fine aggregate is Platte River Valley sand gravel with 1/4 inch maximum size.

Coarse aggregate consists of 1-1/2 inch crushed limestone (3/4 inch for special limited applications) from the Plattsmouth ledge at Weeping Water, Nebraska.

The aggregate materials conformed substantially to ASTM C33, except for minor variations in gradation. The aggregate mix was based on the requirements of the Nebraska Standard Specifications for Highway Construction, modified slightly in accordance with local practice.

Investigation Program

The aggregates for use in the concrete mix were tested for acceptability. The tests were performed by the Omaha Testing Laboratory of Omaha, Nebraska, in accordance with the then current editions of the following ASTM standards:

a. ASTM C40, Organic Impurities in Sands for Concrete;

b. ASTM C75, Sampling Stone, Slag, Gravel, Sand and Stone Block for Use as Highway Materials;

c. ASTM C87, Effect of Organic Impurities in Fine Aggregate on Strength of Mortar;

d. ASTM C88, Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate;

e. ASTM C117, Materials Finer than No. 200 Sieve in Mineral Aggregate by Washing;

f. ASTM C123, Lightweight Pieces in Aggregate;

g. ASTM C127, Specific Gravity and Absorption of Coarse Aggregate;

h. ASTM C128, Specific Gravity and Absorption of Fine Aggregate;

i. ASTM C131, Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine;

j. ASTM C136, Sieve or Screen Analysis of Fine and Coarse Aggregates;

k. ASTM C142, Clay Lumps in Natural Aggregates;

l. ASTM C235, Scratch Hardness of Coarse Aggregate Particles;

m. ASTM C289, Potential Reactivity of Aggregates;

n. ASTM C295, Petrographic Examination of Aggregates for Concrete;

o. ASTM C324, Potential Volume Change of Cement Aggregate Combinations.

Mixes

The design and associated tests of concrete mixes was also performed by the Omaha Testing Laboratory. Concrete strengths were predicated on the basis of ACI-301, "Specification for Structural Concrete for Buildings", Section 308, Method 2 for Ultimate Strength Type Concrete and Prestressed Concrete.

The basic concrete mixes evolved and utilized for construction are shown in Table 5.2-2.

Table 5.2-2 - Concrete Mix Compositions

Application	Containment Mat & Shell	Special Limited Applications*
Aggregate Size, in	1-1/2	3/4
Compressive Strength, psi at 28 days	5000	5000
Water-Cement Ratio, gals, max/sack	4.25	4.00
Cement Content, max sacks/ cu. yd.	6.25	7.00
Aggregate Proportions, %		
Sand Gravel	50 ± 3	50 ± 3
Crushed Limestone	50 ± 3	50 ± 3
Air Content, %	4.75 ± 0.75	4.75 ± 0.75
Pozzolith Low Heat, lbs/ sack cement	0.2	0.2
Slumps, in, maximum	3	3
Density, lb/cu. ft, minimum	145	145

*This mix was a special mix utilizing 3/4 in. maximum crushed limestone for special limited applications where the density of reinforcement and difficulty of ensuring good placement of 1-1/2 in. aggregate concrete warranted its use. Its application has been restricted to the portion of the structure in the immediate vicinity of the anchorages for the helical tendons.

5.2.2 Reinforcing Steel and Cadweld Splices

Reinforcing Steel and Cadweld Splices for the NSSSRP construction opening are described in Section 5.3.6.3.

Reinforcing steel in the foundation mat consists of deformed billet steel bars which conformed to ASTM A615, Grade 60. This steel has a minimum yield strength of 60,000 psi, minimum tensile strength of 90,000 psi, and a minimum elongation of 7 percent in an 8 inch specimen.

Reinforcing steel in the cylindrical wall, the domed roof and around openings is deformed billet steel which conformed to ASTM A615, Grade 40. This steel has a minimum yield strength of 40,000 psi, minimum tensile strength of 70,000 psi, and a minimum elongation of 7 percent in a 8 inch specimen.

Splices for reinforcing steel conformed to the requirements of ACI Code 318-63. Splices for bar sizes No. 11 and smaller were lapped except at certain locations such as at the construction openings. Splicing for bars at these locations and for all bar sizes larger than No. 11 was accomplished by approved positive connectors, specifically Cadweld "T" series connectors, (as manufactured by Erico Products Incorporated of Cleveland, Ohio), designed to develop the specified tensile strength of the reinforcing steel.

No reinforcing steel strength or tack welding was permitted.

5.2.2.1 Reactor Cavity and Compartments and Maintenance Activities

For the reactor cavity and compartments, the yield strength (F_y) of reinforcing steel is based on testing in accordance with ASTM Standards. Historical laboratory test data of Grade 40 steel reinforcement used in the construction of Fort Calhoun Station Class I structures indicate that the yield strength test results exceed the material specification values used in the original design. Statistical analysis of Certified Material Test Reports (CMTR) for the CIS confirms the 95% confidence yield strength for Grade 40 reinforcing steel to be 44 ksi. This is applicable to the reactor cavity and compartments and raises the yield strength from 40 ksi to 44 ksi.

The higher reinforcing steel strength is also used to evaluate the containment internal structure beams, slabs, and columns.

5.2.3 Prestressing Post-Tensioning System

Section 5.3.6.4 describes the prestressing system and the NSSSRP construction opening.

5.2.3.1 Description

The prestressing system is the 90 wire BBRV system as furnished by the Inland-Ryerson Construction Products Company.

Each tendon consists of 90 parallel 1/4 inch diameter high tensile cold drawn, stress relieved wires. Each end of the tendon terminates in an anchorage assembly which transfers the tendon force to the concrete structure. Positive anchorage of the wire ends is achieved with 3/8 inch diameter button heads, cold formed at the wire ends.

The anchorage system consists of:

a) A stressing head on one end of the tendon and a stressing head/shim nut combination on the other end of the tendon: The cold formed button head on the tendon wire end transmits the tendon force to the stressing head. The stressing head or stressing head/shim nut is engaged through the external buttress threads by a hydraulic stressing device which induces the desired force in the tendon.

b) A set of split tube shims: These shims are installed between the stressing head or stressing head/shim nut and the bearing plate; they maintain the strain induced in the tendon and transmit the tendon force from the stressing head to the bearing plate.

c) A bearing plate to transfer the tendon force to the concrete surface.

The tendons were installed in the concrete structure in steel conduits, rigid or semi-rigid depending on location, and fully interlocked and completely sealed from terminal to terminal against leakage of mortar into the conduit.

5.2.3.2 Material Properties

Tendon wire was cold drawn and stress relieved, with a minimum ultimate strength of 240,000 psi. Each 90 wire tendon has a minimum ultimate strength of 1,060,000 pounds and a minimum yield strength of 848,000 pounds.

Anchor components were designed to work within their elastic range up to a load equal to the ultimate strength of the tendon, except for the plastic seating of the button heads.

Governing prestressing material specifications were as follows:

a) Tendon wires: ASTM A421, Type BA;

b) Bearing plate: ASTM A36, (Fine Grained Practice);

c) Anchor head: AISI 1141 Special Quality;

d) Bushing: HFSM Tubing AISI 1045 or AISI 4140;

e) Shims: HFSM Tubing AISI 1026;

f) Tendon sheathing:

   1. Rigid: 4 inch O.D., 14 gage and 3 gage flash controlled welded tubing;

   2. Semi-rigid: 3-3/4 inch I.D. 22 gage spirally formed from steel strip with interlocked seam.

5.2.3.3 Corrosion Protection

Tendons were prepared at the factory for shipping by application of a coating of a thin-filmed temporary corrosion inhibitor. Each tendon was then coiled and enclosed in a plastic bag, and 4 ounces of Shell No. 250 VPI (vapor phase inhibitor) powder was dusted into each bag. Each bag was then sealed and shipped to the job site.

The temporary corrosion inhibitor contained not more than 10 ppm each of chlorides, nitrates and sulfides. It was designed to adhere to the tendon during installation in the structure and to provide corrosion protection under the prevalent field conditions.

The permanent corrosion protection system subsequently applied to the tendons was manufactured and supplied by the same manufacturer and has the same chemical restrictions as the temporary corrosion inhibitor to ensure that all materials used in the compounding of both systems were compatible.

The exposed portion of the bearing plates, which are located in the stressing galleries and receive the tendon stress, were coated with grease after final tendon installation to prevent rusting.

5.2.4 Liner Plate and Penetrations

Liner plate for the NSSSRP construction opening is described in Section 5.3.6.2.

The liner plate is 1/4 inch in thickness. The materials specified for the various components of the containment liner are in accordance with the following:

a. Liner plate at the foundation mat and within the recess below the reactor conformed to ASTM A516, Grade 60, except that Charpy vee-notch impact tests in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Class B vessels, were performed at a maximum temperature of 0°F.

b. Material for the cylinder and dome liner plate, for embedded structural shapes and stiffeners, for test channels, for reinforcing plates around penetrations, for personnel air lock internals and for the polar crane, with exception of the thickened liner plate, conformed to ASTM A36.

c. The thickened liner plate at the crane supports conformed to ASTM A516, Grade 60. In addition these plates meet and in special areas exceeded the requirements of ASTM A435 for ultrasonic testing.

d. Plate for the personnel air lock barrel, bulkhead and doors, and for the equipment hatch barrel and cover and surrounding reinforcing plates, conformed to ASTM A516, Grade 60, manufactured to ASTM A300 requirements, except that Charpy vee-notch impact tests were in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Class B vessels, and were performed at a maximum temperature of 0°F.

e. Material for penetration sleeves conformed to ASTM A333, Grade 1, for seamless pipe and ASTM A516, Grade 60, manufactured to ASTM A300 requirements for pipe formed from rolled plate, except that Charpy vee-notch impact tests were in accordance with the ASME Boiler and Pressure Vessel Code Section III, Class B vessels, and were performed at a maximum temperature of 0°F.

f. Bolting materials for the equipment hatch cover conformed to ASTM A320.

g. Material for forged flanges conformed to ASTM A350 except for impact test requirements; material for flanges fabricated from plate conformed to ASTM A516, Grade 60, manufactured to ASTM A300 requirements. Charpy vee-notch impact tests were in accordance with the ASME Boiler and Pressure Vessel Code Section III, Class B vessels, and were performed at a maximum temperature of 0°F.

h. Pipe for pressure connections to the test channels over liner seam welds inaccessible for inspection conformed to ASTM A106, Grade B.

Materials for penetrations and openings which must resist full design pressure conformed to the requirements of the ASME Boiler and Pressure Vessel Code, Section III, Class B vessels. NDT considerations were accounted for in paragraph N-1211a, ASME Boiler and Pressure Vessel Code, Section III, which required that Charpy V-specimens be tested at a temperature at least 30°F lower than the lowest service metal temperature in accordance with the requirements of paragraphs N-331, N-332, N-511.2 and N-515. The lowest service metal temperature for penetrations within the containment is 50°F. All those penetrations projecting outside the containment are located within the enclosure provided by the auxiliary building; the lowest service metal temperature for these penetrations is 50°F.

5.2.5 Protective Coatings and Paints Inside the Containment

Protective coatings for the NSSSRP construction opening are described in Section 5.3.6.2.

Construction

Coatings used within the containment were specified to withstand accident conditions. The applicable requirements of ANSI N101.4-1972, Quality Assurance for Protective Coatings Applied to Nuclear Facilities, and Regulatory Guide (RG) 1.54, Revision 0, Quality Assurance Requirements for Protective Coatings Applied to Water Cooled Nuclear Power Plants, were implemented for modification activities and meet or exceed original plant specifications and manufacturer recommendations.

Most carbon steel original equipment has prime coats of Carbozinc or Carboweld and finish coats of Phenoline 305, all products of the Carboline Corporation. The Safety Injection Tanks were primed with DuPont Corlar 825-8031 epoxy zinc chromate primer number 583 while the Pressurizer Quench Tank was primed with DuPont number 773 Dulux zinc chromate primer. Documentation that all primers and top coats stated above have been tested or are acceptable for their service applications is shown by Carboline Corporation and DuPont specifications and test sheets.

Concrete inside the containment was treated with Amercoat Nu-Klad 1100A and top coated with Amercoat No. 66. Testing has proven this type of coating to be adequate under accident conditions.

Some minor components such as instrument housings and electrical cabinets have proprietary coatings, mostly baked enamels. The total area so coated, however, is very small compared to the total surface area.

Maintenance and Modification

Coating materials currently used for maintenance of existing surfaces or for surfaces of new modifications installed inside the reactor containment are epoxy materials tested to withstand Fort Calhoun design basis accident (i.e., loss of coolant accident (LOCA)) conditions. The coating materials and their application comply with the requirements of RG 1.54. Each coating was tested in accordance with ANSI N101.2, Protective Coatings (Paints) for Light Water Nuclear Reactor Containment Facilities, or ASTM D3911, Evaluating Coatings Used in Light-Water Nuclear Power Plants at Simulated Design Basis Accident (DBA) Conditions, to conditions that bound the Fort Calhoun DBA conditions. Prior to exposure to the simulated DBA conditions, each coating test panel was irradiated to an accumulated dose of at least 1.8×10^8 Rads in accordance with ASTM D4082, Standard Test Method for Effects of Gamma Radiation on Coatings for Use in Light-Water Nuclear Power Plants.

Use of the above coatings precludes the possibility of large flaking or skinning off of sheets of paint during accident conditions. Since the only way use of these coatings could affect performance of engineered safety features is by flaking off and entry through the drains during recirculation after an accident, it is expected that their use will have no deleterious effects on engineered safety equipment and that flow blockage or fouling of heat transfer surfaces will not take place.

5.2.6 Compressive Test Strength of Concrete

Specified compressive concrete strength (f_c') is used for design of Class I structures except for the structures defined below where actual compressive strength has been established.

Where applicable, actual concrete design compressive strength is established using analysis of 28-day test data based on Building Code ACI 318-63 Section 504(c).

For structures designed in accordance with Part IV-A of ACI 318-63, 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 structures designed in accordance with Part IV-B of ACI 318-63, 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

A rolling average is calculated for consecutive strength tests, and the minimum strength of the average values represents the area specified. The design strength is the lesser of the minimum rolling average design basis concrete strength or the 95% confidence level of all test data for the structural group.

Concrete strengths based on 28-day test data may only be used in areas not potentially affected by degradation of concrete by long-term exposure to high radiation, damaging chemicals, or excessive moisture.

Actual concrete design compressive strengths are established using the methodology described above in the following areas:

- Auxiliary Building: Except for exterior walls below elevation 1007 ft.

- Containment Internal Structures: Except for the concrete walls surrounding the reactor vessel and the floor of the reactor cavity.

Actual concrete design compressive strength is not used for the Intake Structure or its foundation, the Spent Fuel Pool, Containment Structure or the Containment / Auxiliary Building foundation.

USAR-5.11

Structures

Structures Other Than Containment

Rev 14

Safety Classification:

Safety

Usage Level:

Information

Change No.:	
Reason for Change:	
Preparer:	
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Fort Calhoun Station

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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

The allowable stress capacity of Class I structural steel is determined in accordance with the allowable stress design provisions from the AISC Code, 1963 edition.

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 concrete structures were originally designed for operating basis conditions using ACI 318-63 Code working stress capacities as discussed below in Section 5.11.3.4(b)

a. Ultimate Strength Design

With the approval of Amendment No. XXX, the design criteria for operating basis conditions changed to implement the ultimate strength design method for normal/operating service conditions for changes and reanalysis using the following load combinations:

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

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

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

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

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.

b. Working Stress Design

Foundations for Class I structures are not upgraded to ultimate strength design by Amendment XXX. They are designed on the basis of working stress for the following load combinations:

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

where:

S = Required section capacity

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

The application of limit design method for RC&C walls includes deformation limit equations in ACI 349-97 Commentary on Appendix C Section C.3.

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.

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

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

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

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

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

Φ = Reduction factors as shown in Section 5.11.3.4.

The application of limit design method for RC&C walls includes deformation limit equations in ACI 349-97 Commentary on Appendix C Section C.3

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. Dynamic increase factors appropriate for the strain rates involved for impulsive loads to the reactor cavity and compartment walls and transfer canal comply with ACI 349-97, Appendix C. 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 Sections 5.11.3.1a and 5.11.3.6.

5.11.3.8 Concrete Compressive Strength

Where applicable, concrete design compressive strengths (f_c`) may be established using analysis of historical test data as described in Section 5.2.6.

5.11.3.9 Steel Reinforcing Capacity

Where applicable, steel reinforcing capacity allowable stress values may be established using analysis of historical test data as described in Section 5.2.2.1.

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.