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10 CFR 50.90

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

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

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

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

This 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 shell, the spent fuel pool, and the foundation mat under the containment and auxiliary buildings, as well as the foundation mat under the intake structure. 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 shell, the spent fuel pool, and the foundation mats.
2. Use 28-day test strength of concrete (i.e., not age hardening) where test data indicates material strengths are higher than original material specifications at select locations in the Class I structures.
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. Add new methods for evaluating the concrete in the RC&C walls. These new methods include the limit design method and the use of dynamic increase factors (DIF) for concrete analysis. These methods are applied to the RC&C in combination with replacing WSD with USD, use of a higher rebar (i.e., reinforcing steel) yield strength, and concrete 28-day test strength.
5. Minor clarifications include adding a definition of control fluids to the dead load section. Higher allowable stress values for steel are being added when earthquake loads are applied to the load combination equations.

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 as it is an integral part of meeting OPPD's commitment to restore the design margin for containment internal structures during the fall 2016 refueling outage. 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 August 31, 2015.

Respectfully,

Louis P. Cortopassi
Site Vice President and CNO

LPC/JAC/mle

Enclosure: OPPD's Evaluation of the Proposed Change

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OPPD's Evaluation of the Proposed Change

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

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 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 shell, the spent fuel pool (SFP), the foundation mat beneath the containment and auxiliary buildings and the foundation mat beneath the intake structure. The containment shell 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 more detail in Section 2.0:

1. Apply the ACI 318-63 USD methodology to normal operating/service conditions in lieu of ACI 318-63 WSD methodology currently specified in USAR Section 5.11.

2. Use 28-day test compressive strength of concrete (not age hardening) in accordance with ACI 318-63.

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 limit design and dynamic increase factors (DIF) for the reactor cavity and compartment (RC&C) walls:

   - Use the principles of limit design method and dynamic increase factors where differential pressures and seismic loads are applied to the structures. These methods are used when considering load redistribution while limiting localized deformation for concrete evaluations for the applied loads.
   - In addition to these methods, the proposed license basis changes are also applied to the RC&C and include use of USD versus WSD load combinations, 10% higher steel yield strength, and concrete 28-day test compressive strength instead of the specified strength of 4000 psi.

5. Minor clarifications include adding a definition of control fluids to the dead load section. Higher allowable stress values for steel are added provided when earthquake loads were included in the load combination equations.

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 state-of-the-art computer codes 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. The USAR requires the CIS to meet the more restrictive of two ACI

318-63 design criteria: (1) working stress and (2) ultimate strength. Each of the Class I structures have been evaluated and is operable. The USAR specifies the USD method to ensure no loss of function. 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.

Overall characterization of the changes are minor adjustments to the COR and CLB. These adjustments include using USD replacing WSD and increase in applicable material strengths within the COR. The changes include adding state of the art methods where the COR is silent.

Changing WSD methodology to USD methodology is the most substantial change within the COR that is needed to implement the other changes. The other changes act independently or in combination to address different issues where the design basis is not met. Concrete strength helps in shear conditions, whereas the steel reinforcement strength increase helps in moment conditions. The additional methods are used in cases where there are significant dynamic loads.

Tests performed during construction justify increasing material strength. Materials used in the construction of the plant have significantly higher strengths than what was specified. This does not require a code change.

A description of the Class I structures affected by this LAR is as follows.

Containment

The containment building is a partially prestressed, reinforced concrete, Class I structure composed of a cylindrical wall (i.e., shell), domed roof 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 a ¼” internal carbon steel liner.

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

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.

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 shell, 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)”.

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 shell, spent fuel pool (SFP), and the foundation mats for the containment, auxiliary, and intake structure buildings).
Margin	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. These requirements include all the phi factors identified in the code for concrete and reinforcement steel to provide sufficient margin. 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 state of the art 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 at nuclear power plant design basis. The USD method provides an appraisal of the concrete strength with higher accuracy for evaluating the loads applied to the concrete structures. The USD method is a superior method for reinforced concrete design and is endorsed by the American Concrete Institute in all later revisions of the ACI 318 Code.
Consequences	This proposed change when combined with the change for increased concrete strength described below is expected to eliminate the need to install approximately 12 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 Increase Concrete Compressive Strength (Use historical test data for design basis) – See Section 2.2 for Additional Information

The PLB for concrete strength is to use the core pour test data from the original construction for design basis. This LAR is not proposing to use age hardening but actual test data when the plant was constructed.

Current Licensing Basis	Concrete is currently evaluated using the design value from the original construction specifications (i.e., f_c` = 4000-psi).
Proposed Licensing Basis	When the plant was built, three (3) test samples were obtained from each batch of concrete. The PLB increases the concrete strength from 4000-psi to the lowest value of all historical laboratory-cured test data for pours in selected areas of interest (3 test samples/pour). Historical laboratory testing of these samples shows that the actual compressive strength of the concrete exceeds the original design value (i.e., f_c` = 4000-psi). This application will be limited to select areas in the containment and auxiliary buildings where the concrete is not exposed to high radiation, excessive moisture, or harsh chemicals for prolonged periods. The tests specified in the ACI 318-63 Code are standard tests that would be performed for all nuclear power plants including plants built to today's standards.
Where Applicable	As shown in the Table in Section 2.2 below.
Margin	The tests completed during construction of the plant in the early 1970's show the concrete is stronger than what was specified in the original design, which results in a large increase in structural capacity. The PLB takes advantage of the increase in capacity and increases the design basis value to the most conservative of the three laboratory tests performed for each batch. Adequate design margin is assured because the lowest value of the three (3) historical test samples will be utilized in the design basis analysis. The concrete materials are superior to the concrete specified and used in the design basis. This is discussed in detail in Section 2.10
Consequences	See 2.0.1 above.

2.0.3 Increase Reinforcing Steel Yield Strength (Use historical test data for design basis) – See Section 2.3 for Additional Information

The proposal for steel reinforcement yield stress is to use a statistical increase from 40 ksi to 44 ksi (10%). The use of test data for steel was included in the operability calculations and reviewed during the 2012 inspections.

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	The increase is limited to 44 ksi. Historical test data shows that the mean yield strength with one standard deviation and 95% confidence level supports the 44 ksi. Design margin is maintained because the new value is significantly lower than historical mean yield strength. The historical test data supports an increase of 10% for reinforcing steel yield strength.
Consequences	This PLB change in combination with the changes discussed above (i.e., WSD to USD, concrete strength increase etc.) minimizes the need for physical modifications to CIS and is anticipated to eliminate the need for approximately 8 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 is an arbitrary number to increase the load and 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. The margin is enhanced since the phi factor application accounts for issues that do not exist with the CIS. The reinforcing steel materials are superior to the materials specified and used in the design basis. This is discussed in detail in Section 2.10.

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 add two new methods that include dynamic increase factors (DIF) and limit design method (ductility) applied to the RC&C.

Current Licensing Basis	The CLB assumes WSD methodology, 4000-psi concrete and 40 ksi reinforcing steel.
Proposed Licensing Basis	The PLB adopts the changes listed above (i.e., USD, historical test data for concrete and concrete reinforcement yield strength) and two additional methods (i.e., dynamic increase factors (DIF) 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 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 pressure and seismic dynamic events.
Where Applicable	RC&C portion of the CIS.
Margin	For the use of USD, concrete strength, and yield strength of reinforcing steel, design margin is maintained as described in 2.0.1 through 2.0.3 above. For the use of DIF and limit design methods, design margin is maintained by the conservatism in the load combination equations for short-term high differential pressure. The dynamic effect of differential pressure lasts less than 30 seconds, and the load combinations assume a major earthquake concurrent with a design basis accident, which is extremely conservative and supports the use of DIF and the limit design method for dynamic loads. There is significant use of these methods in other nuclear power plants (See Section 4.2).
Consequences	The limited clearances available in the RC&C make the placement of additional concrete structures in that area extremely difficult and unwarranted given that structural analysis shows that adequate design margin exists with the proposed changes. 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 Minor changes to the USAR – See Sect. 2.5 for Additional Information

The LAR proposes some minor changes that include adding the definition of controlled fluids and increases in allowable stress values for steel design.

Current Licensing Basis	The CLB has no discussion of hydrostatic load from controlled fluids (i.e., fluid in tanks) or steel design allowable increases for the OBE case. There is a phi factor for steel design in the MHE case up to 0.9 on the load side of the combination equations.
Proposed Licensing Basis	The PLB adopts controlled fluids as a dead load and includes allowable stress values for steel design.
Where Applicable	All Class I structures.
Margin	These changes are considered clarifications that do not impact design margin. They are called out here because they are incorporated in the proposed USAR change, but otherwise do 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. The increase in steel allowable stress values for OBE is defined in the American Institute of Steel Construction (AISC)-63 Code, which is the COR for steel design. The allowable stress value of 0.9 yield is in the CLB as a phi factor.
Consequences	These changes support the distribution of loads for controlled fluids and have minimal impact on restoration of the design basis of CIS or the auxiliary building.

In summary, these changes 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 necessary to restore the design basis of CIS.

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 capacity 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:

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

Where:
S = Required section capacity
U = Required ultimate strength capacity
D = Dead load, including internal controlled hydrostatic
L = Live load
H = Soil Load
W = Wind load
E = Design earthquake
F = Flood hydrostatic load to elevation 1007 feet
Φ = Capacity 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 (Reference 6.13) and remained constant for more than thirty years. In addition, soil pressure (H) (lateral earth pressure or ground water pressure for design of structures below grade) is added to align with editions of ACI 318 issued subsequent to 1963.

(2) The USD load combinations are developed for normal operating/service conditions using the current ultimate strength method USAR equation (i.e., 1/Φ (1.0 D ± 0.05D + 1.25Pc + 1.25E + 1.0Tc)) for the extreme condition, no loss-of-function equation. The equation is modified by replacing the Pc load (differential pressure between compartments resulting from reactor coolant system break) with the live load (L) and replacing the 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.

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 shell, 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 Definition of Concrete Compressive Strength per the ACI 318-63 Code

The containment shell and the foundation mats were originally designed using a compressive strength of concrete (f_c\`) = 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 a compressive strength of concrete ($f_c$\`) = 4000-psi. No change is proposed to the design criterion for compressive strength of concrete in the containment shell, intake structure or foundation mats.

As shown in the table below, the proposed change will allow reinforced concrete structures located in specified areas of the auxiliary or containment buildings to be designed and evaluated using concrete compressive strength based on historical laboratory test data of cylinder break strengths as their design basis instead of the original design value (i.e., f_c\` = 4000-psi). This is appropriate as testing shows that the actual compressive strength of the concrete exceeds the original design value.

The increased f_c\`, based on original test data, does not include age hardening. As defined in Section 301 of ACI 318-63, “Compressive strength shall be determined by test of standard 6 x 12-in. cylinders made and tested in accordance with ASTM specifications at 28 days or such earlier age as concrete is to receive its full service load or maximum stress.” The ACI 318-63 Code is the CLB COR for Class I structures at FCS. Historical laboratory test data from several FCS Class I structures indicate that 28-day compressive strength test results exceed the value of 4000-psi used in the original design by approximately 500 to 1500-psi (i.e., approximately 4500 to 5500-psi).

The concrete design compressive strengths (f_c\`) may be established using analysis of historical laboratory test data based on the minimum compressive strength test result of all samples environmentally controlled in the laboratory. This approach is in alignment with the methodology of ACI 318-63, Chapter 5 albeit more conservative.

This definition of f_c\` based on historical laboratory test data 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 in the auxiliary building and the containment building for containment internal structures where concrete compressive strength historical laboratory test data may and may not be used are shown in the following table.

Area	Allowed Application of Historical Test Data	Non-Allowed Application of Historical Test Data[1]
Auxiliary Building	Elevations ≥ 1007 feet.	Elevations < 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	Not allowed.	Not applicable, will continue to use f_c` = 4000-psi.
Containment Shell / 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 1: The application of historical laboratory test data for concrete compressive strength in lieu of design values is not allowed where structures undergo prolonged exposure to high radiation, excessive moisture, or harsh chemicals.

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

2.3 Definition of Reinforcing Steel Yield Strength per the ACI 318-63 Code

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. The original operability evaluated 506 samples that were used throughout the structures and the results using 1.34 standard deviation of the mean yield strength is 44.1 ksi. The design basis will be based on reinforcing steel and the testing of the representative samples used specifically for the CIS. Quality records show that there were 101 heat code samples used in the construction of the CIS. Some of the heat code yield stress values could not be identified and 87 of the 101 samples are known. Based on 87 samples specifically used for CIS, the mean yield strength (μ) was determined to be 50.29 ksi. With one standard deviation (σ) = 3.84 ksi, the final steel yield strength using $\mu - 1.34\sigma$ = 45.15 ksi. Using the 95% confidence level of $\mu - 1.65\sigma$ = 43.95 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).

2.4 Limit Design Method, Use of Dynamic Increase Factors, and Higher Reinforcing Steel Yield Strength for Reactor Cavity and Compartments

The change in the FCS CLB is to use the limit design method for concrete analysis and evaluation in the RC&C as defined in ACI 349-97, Appendix C, “Special Provisions for Impulsive and Impactive Effects” for special condition evaluations.

In addition to ductility, this change will allow the use of DIF methodology when there are impulsive loads present for the reactor cavity and compartment walls. This applies to

the no loss-of-function conditions as defined in USAR Section 5.11 and does not affect the normal operating/service conditions.

This change to use the DIF methodology 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 higher differential pressures.

Use of F_y of reinforcing steel to be based on testing in accordance with American Society for Testing and Materials (ASTM) Standards.

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

2.5 Additional Clarifications within USAR Section 5.11

Additional clarifications and formatting updates are proposed for incorporation into USAR Section 5.11 to describe the structural design criteria of Class I structures other than containment. Clarifications of note are:

a. 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.
b. While the Fort Calhoun structures are primarily reinforced concrete construction, the structural design criteria will now separately define the licensing basis requirements for structural steel. Clarity was added to address seismic load conditions. For operating basis, the allowable stresses can be increased by 1/3 when the load combinations include wind or seismic load(s). This is defined in American Institute of Steel Construction (AISC)-63, (Reference 6.20, Section 1.5.6). For design basis, the allowable stresses can be increased by 50% to a maximum of 90% of yield. This is included in the original license basis where the capacity strength reduction factors (Φ) for steel was 0.9. Thus, the load was increased by 1/0.9. The equivalent reciprocal is applied to the capacity of the structural elements instead of the load side for steel. Whereas the load is not increased by a factor of 1/0.9, but the capacity of the steel is reduced by a factor of 0.9.
c. A typographical error is corrected at the end of Section 5.11.3.7 where the reference should be to Section 5.11.3 and not to Section 5.11.3.1a.

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 lack of refinement in the WSD method gives excessively conservative results for member flexural capacity than is obtained using the USD method. This unnecessary conservatism also exists for axial and shear capacities. 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 shell or the foundation mat, 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 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, U.S. commercial nuclear plants do not commonly use the ACI 318-63 Code load combination (i.e., 1.5D +1.8L).

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 are a simple conversion to consider loads in combination with the flood load (F).

2.6.2 Definition of Concrete Compressive Strengths per the ACI 318-63 Code

The original design of Class I structures other than the containment shell and the foundation mats used a 4000-psi specified concrete compressive strength.

The use of an increased concrete compressive strength as justified by historical laboratory test data 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. Thus, concrete strength design values will be based on data from the 28-day historical laboratory test data (not related to age hardening) in lieu of f_c` = 4000-psi.

As defined in Section 301 of ACI 318-63, "Compressive strength shall be determined by test of standard 6 x 12-in. cylinders made and tested in accordance with ASTM specifications at 28 days or such earlier age as concrete is to receive its full service load or maximum stress."

2.6.3 Definition of Reinforcing Steel Yield Strength per the ACI 318-63 Code

The use of 10% higher yield strength for reinforcing steel is limited to approximately 1.65 standard deviations of the Certified Material Test Reports (CMTR's), 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

Section 2201 of ACI 318-63 entitled "Structural design of walls" states that the limits of thickness and quantity of reinforcement shall be waived where structural analysis shows adequate strength and stability. The finite element model used to analyze the RC&C demonstrates this requirement is met. Such a system may be designed by any procedure satisfying conditions of equilibrium and geometrical compatibility if it is shown that the design strength at every section is at least equal to the required strength, particularly concrete strain limits, and all limiting conditions including deflection limits are met. When the most stressed sections of the structure reach yield moment, they tend to maintain their moment capacity with further increase in curvature under applied loads while yield of the reinforcement spreads to other sections of the structure. Limit analysis computes the ultimate load on the structure and distribution of the moments and shears at ultimate load assuming the sections are sufficiently ductile to enable the required redistribution of bending moments to occur. Ductility limits are enforced through analytical checks of the strain and deflection.

2.6.5 Controllable Fluids and Seismic Factors for Steel Design

The clarifications associated with fluids are consistent with the standard approach to addressing tanks and cavities that contain fluids. The seismic factors are within the original license basis where the AISC-63 addresses the allowance of increasing the capacity of steel for seismic for normal/service conditions operating basis earthquake (OBE). The allowance for the no loss of function (MHE) is within the license basis where

the capacity strength reduction factors (Φ) for steel is 0.9. There is no change to the license basis.

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. The results of 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."

2.8 Analysis Supporting the License Amendment Changes

Analysis is in progress to restore the CIS to design basis. The PLB change has been evaluated with preliminary results that support the effects of the license basis changes to show that the structure is safe and can be restored to design basis with minor adjustments to the current license basis as requested in this LAR.

The table below shows the preliminary results from the analysis performed to date while implementing the changes to the license basis compared to the design basis calculation prepared in 2012.

Table 1: 13 beams on 1013' elevation that have IR's greater than 1.0 and considered for possible modifications.

<table>
<tr><th rowspan="2">Beams on 1013' Elevation</th><th colspan="2">Current License Basis (CLB)</th><th>Proposed License Basis (Preliminary)</th><th rowspan="2">Notes (beams with IRs > 1.0 per CLB that do not need additional potential modifications in addition to the potential modifications to the beams listed in the first column).</th></tr>
<tr><th>WSD IR</th><th>USD IR</th><th>USD IR with new load factors</th></tr>
<tr><td>B5-13</td><td>1.04</td><td>0.81</td><td>0.80-0.90</td><td></td></tr>
<tr><td>B13-13</td><td>1.27</td><td>0.86</td><td>0.70-0.80</td><td>B-42b overstress condition is addressed by this area under the equipment hatch.</td></tr>
<tr><td>B14-13</td><td>0.82</td><td>0.73</td><td>NA</td><td rowspan="3">This beam B14-13 would be reinforced with new column to addresses over stress conditions in B43-13 and B44-13.</td></tr>
<tr><td>B43-13</td><td>1.30</td><td>1.00</td><td>0.70-0.80</td></tr>
<tr><td>B44-13</td><td>1.48</td><td>1.09</td><td>0.80-0.90</td></tr>
<tr><td>B21-13</td><td>1.46</td><td>1.44</td><td>1.20-1.30</td><td rowspan="6">These beams address over stress conditions in B32-13, B35a-13, B35b-13, B39a-13, B39b-13.</td></tr>
<tr><td>B22a-13</td><td>1.65</td><td>2.47</td><td>1.20-1.30</td></tr>
<tr><td>B22b-13</td><td>1.66</td><td>2.44</td><td>1.20-1.30</td></tr>
<tr><td>B23-13</td><td>1.65</td><td>1.43</td><td>1.05-1.15</td></tr>
<tr><td>B36-13</td><td>1.30</td><td>2.17</td><td>0.85-0.95</td></tr>
<tr><td>B46b-13</td><td>1.05</td><td>1.19</td><td>0.80-0.90</td></tr>
<tr><td>B27-13</td><td>1.12</td><td>0.92</td><td>0.70-0.80</td><td rowspan="2">These beams address over stress in B3-13, B17-13, B18-13, and B28-13.</td></tr>
<tr><td>B29-13</td><td>2.45</td><td>2.38</td><td>0.80-0.90</td></tr>
</table>

Table 2: 4 beams on 1045' elevation that have IR's greater than 1.0 and considered for possible modifications.

Beams on 1045' Elevation	Current License Basis (CLB)		Proposed License Basis (Preliminary)	Notes (beams with IRs > 1.0 per CLB that do not need additional potential modifications in addition to the potential modifications to the beams listed in the first column)
	WSD IR	USD IR	USD IR with new load factors	
B12-45	1.90	1.75	0.90-1.00	Beam above equipment hatch
B21a-45	0.65	0.44	0.30-0.40	These beams are considered for modification to transmit load from 1060' elevation down to the 1013' elevation.
B21b-45	0.65	0.44	0.30-0.40	
B23-45	1.32	0.81	0.75-0.85	

Table 3: 5 beams on 1060' elevation that have IR's greater than 1.0 and considered for possible modifications.

Beams on 1060' Elevation	Current License Basis (CLB)		Proposed License Basis (Preliminary)	Notes (beams with IRs > 1.0 per CLB that do not need additional potential modifications in addition to the potential modifications to the beams listed in the first column)
	WSD IR	USD IR	USD IR with new load factors	
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. No modifications were used in the model that resulted in the IR's presented in this table.
2. LAR methodologies 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 CLB calculation uses a conservative vertical dynamic analysis of the building which is beyond the criteria in the USAR.

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 the limit design and DIF methods to the containment internal structure RC&C.

ACI 349-97 is based on ACI 318-89 (Revised 1992) [aka "Building Code"]—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 progressive 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.

A matrix was created to summarize the dissimilarities and to provide a general disposition regarding the as-built Fort Calhoun Station (FCS) Class I structures. The matrix is not included in this submittal but is summarized as follows:

The differences between the ACI 318-63 USD methodology and the ACI 349-97 methodology were reviewed and evaluated to determine the appropriateness of adopting ACI 349-97, Appendix C for impactive and impulsive loadings for the RC&C walls. The main differences are summarized as follows:

- Provisions for moment redistribution (ACI 349-97, Chapter 8)—ACI 349-97 allows for 20 percent moment redistribution which bounds the 10 percent moment redistribution allowed by ACI 318-63.
- 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.
- Reinforcement detailing (ACI 349-97, Chapters 7 & 12)—ACI 349-97 expands on provisions for adequate reinforcement detailing, including introducing an excess reinforcement adjustment ratio for development length computation as well as the classification of lap splices of deformed bars (i.e. Classes A and B). OPPD takes exception to the newer Code detailing due to the time of construction.

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.

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 on the structural re-evaluation of the existing FCS concrete structures and have been reconciled. The Comparison Matrix reports on the findings and the results of the disposition.

2.9.4 Conclusions

Based on the comparison of ACI 349-97 Code and the dissimilarities to the ACI 318-63 "Building Code," it has been determined that use of the provisions of ACI 349-97 would be advantageous for re-evaluation of the RC&C. Use of the USD methodology is adequate to evaluate the RC&C. The evaluation concludes that the use of the ACI 349-97 methods maintain the safety margin.

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 very detailed procedures for controlling installation of steel reinforcement and the concrete. Every delivery of these materials required testing and records identifying their placement within the Class I structure. The records were maintained and transmitted to OPPD (Reference 6.26) and include signoffs for each delivery, steel reinforcement test data, and the general location where the steel reinforcement was placed in the Class I structures. Numerous concrete test (i.e., core pour) records are available that accurately show test results and reference the specific 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 very important 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 rigorously 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. 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 1960's. 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 original FCS structural design criteria for Class I structures already 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 new 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 factors are less severe 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.

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 Definition of Concrete Compressive Strengths per the ACI 318-63 Code

The request to implement the ACI 318-63 Code's definition of concrete compressive strength (f_c') is required because the original design basis calculation used a generic 4000-psi compressive strength. The material strength increases above design values apply 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 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 concrete compressive strength.

The ACI 318-63 Code is the CLB COR for the design and construction of reinforced concrete structures at FCS. Full and detailed records from concrete pours at FCS as well as historical test data results are available and the data supports an increase in concrete compressive strength above the 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 increased design value will be established using analysis of historical lab test data, based on the minimum compressive strength test result of all samples environmentally controlled in the lab.

For additional conservatism, application of the increased design value will be limited to areas in the auxiliary building and containment internal structures that do not expose the concrete to excessive moisture, high radiation, or harsh chemicals for prolonged periods.

3.3 Definition of Reinforcing Steel Yield Strength per the ACI 318-63 Code

The use of higher steel reinforcement yield strength is limited to the RC&C in all Modes and to CIS. There is considerable conservatism in the load combination equations where the differential pressure of an accident is added to an earthquake load. The differential pressure from the accident lasts long enough to exert a pressure on the concrete, which is significantly reduced after 30 seconds. In such a situation (i.e., design basis accident concurrent with seismic event), there is still design margin both in the concrete and with steel reinforcement yield strength elevated to 44 ksi. After 30 seconds, the pressure differential subsides, the pressure in containment equalizes, the loads drop, and design margin increases. The higher steel yield for the RC&C has negligible effect on design margin. The higher steel yield strength for CIS has minimal effect on design margin and as it is limited to use, no effect on the ability to achieve safe shut down.

3.4 Limit Design Method, Dynamic Increase Factor for Impulsive Loads, and Higher Reinforcing Steel Yield Strength for Reinforced Concrete Design and Analysis of Reactor Cavity and Compartments

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. For impactive and impulsive effects, due to the extreme nature of these loading conditions, the acceptance criteria is expanded to use a ductility approach consistent with ACI 349-97, Appendix C for the reactor cavity and compartment walls. All applicable load combinations involve the compartmental pressure loads for the high-energy line break loading case. Ductility limits are as specified in Sections C3.3, C3.4, C3.5 and C3.7 of ACI 349-97, Appendix C. ACI 349-97, Appendix C is a commonly accepted design code used in the Nuclear Industry.

Furthermore, this application of the limit design method and DIFs are 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.

The request to implement a design method that allows inclusion of dynamic strength increase factors (DIF) is required because there are no equivalent factors in the ACI 318-63 Code. OPPD requests that DIFs may be utilized as specified in ACI 349 C.2.1. 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. The other provisions of ACI 349-97 Appendix C are applied to American Society for Testing and Materials (ASTM) A15 rebar, used at FCS, similarly as permitted for ASTM A615 rebar and serve to insure that the structure has adequate ductility and reserve strength to withstand the imposed ductility demand. The ACI 349-97 Code was published well after ASTM A15 was withdrawn and so did not address ASTM A15 rebar. ACI 349-97, Appendix C is a recognized code used in the Nuclear Industry.

4.0 <u>REGULATORY EVALUATION</u>

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. Criterions 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 pro0duced 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 considered a safe and superior method for reinforced concrete design and is endorsed by the American Concrete Institute in all later revisions of the ACI Code. Full and detailed records from concrete pours at FCS as well as historical test data support the use of 28-day test compressive strength of concrete (not age hardening) in lieu of the current design value of 4,000-psi. Application of the new design value based on the 28-day test 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 maintains design margin and its use in design basis calculations is restricted to the RC&C and to the CIS.

Inclusion of the Limit Design Method and dynamic strength increase factors (DIF) fill 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 a commonly accepted design code used in the Nuclear Industry.

4.2 Precedent

The ACI 318-63 Code was commonly 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 a 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 the limit design method to evaluate reinforced concrete using ACI 349-97, Appendix C - Special Provisions for Impulsive and Impactive Effects is a method previously endorsed by the NRC for impulsive loads in SRP 3.8.4 and RG 1.142. This criterion is similar and appropriate for the compartmental pressure and high-energy line break load conditions at FCS.

The 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. Material strength increases above design values apply to concrete compressive strength (f_c') (not related to age hardening) as defined in Section 301 of 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 shell, the spent fuel pool (SFP), and the foundation mats under the containment building, auxiliary building, and the intake structure. These structures will continue to utilize the most conservative requirements obtained by comparison of the results of three independent methods of design including the working stress design (WSD) method of the American Concrete Institute (ACI) 318-63 Code 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 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. The 10% higher steel yield for the reactor cavity and compartment (RC&C) and containment internal structures (CIS) has negligible effect on design margin.

Therefore, the proposed change does 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 use the limit design method and dynamic increase factors (DIF) of ACI 349-97, Appendix C will demonstrate that the concrete structures meet required design criteria. Defining the compressive strength of concrete (f_c`) as the 28-day compressive test (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 effect on design margin for the RC&C or the CIS.

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 shell, 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 and therefore, new designs or re-evaluations of existing Class I structures must still prove capable of coping with design basis loads.

Redefining the compressive strength of concrete (f_c`) as the 28-day test (not age hardening) is justified based on historical test data and further constrained by limiting its application to areas where the concrete is not exposed to harsh conditions. ACI 349-97, Appendix C is a commonly accepted design code used in the nuclear industry. Calculations applying the limit design method, inelastic energy absorption method and DIFs of 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 historical test data and has minimal effect on design margin for the RC&C or the CIS.

Therefore, the proposed change does 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 NUREG/CR-0098, “Development of Criteria for Seismic Review of Selected Nuclear Power Plants,” N.M. Newmark and W.J. Hall, May 1978

6.16 EPRI NP-6041, Revision 1, “A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1),” August 1991

6.17 “Seismic Design, Assessment and Retrofitting of Concrete Buildings,” Fardis, M., Springer, New York, 2009

6.18 “On the Post-Peak Ductility of Shear-Critical Beams,” Vecchio, F.J., ACI Special Publication Vol. 237 pp.109-128, 2006

6.19 ASCE 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities”

6.20 AISC-63, American Institute of Steel Construction, Inc.

6.21 Bechtel Power Corporation, Topical Report BC-TOP-9A, “Design of Structures for Missile Impact,” Revision 2, September 1974

6.22 Letter from NRC (J. M. Sebrosky) to OPPD (L. P. Cortopassi), “Fort Calhoun Station, Unit No.1-Issuance of Exigent Amendment [272] Re: Revise Current Licensing Basis for Addressing Design-Basis Tornado/Tornado Missile Impact (TAC No. MF2469),” dated July 26, 2013 (NRC-13-0095) (ML13203A070)

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

6.24 NUREG/CR-3805, “Engineering Characterization of Ground Motion – Task 1, Effects of Characteristics of Free-Field Motion on Structural Response,” R.P. Kennedy, et al., May 1984, and NUREG/CR-4826, “Seismic Margin Review of the Main Yankee Atomic Power Station,” P. Prassinos, R. Murry, G. Cummings, Lawrence Livermore National Laboratory Report UCID-20948, March 1987

6.25 Quality Assurance Program (WIP 008372).

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

6.27 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

Safety Classification: Safety

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Change No.:	
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Fort Calhoun Station

Table of Contents

List of Tables

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 to be based on testing in accordance with ASTM Standards. Historical lab 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. The yield strength for Grade 40 reinforcing steel may be determined from Certified Material Test Reports (CMTR), i.e., historical lab test data, in accordance with applicable ASTM standards. 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 may also be 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

Historical lab test data for several Class I Structures indicate that 28-day compressive strength test results exceed the material specification values used in the original design.

Where applicable, concrete design compressive strengths (f_c') may be established using analysis of historical lab test data based on the minimum compressive strength test result of all samples environmentally controlled in the lab. This approach is in alignment with, but more conservative than the methodology of ACI 318-63, Chapter 5. Concrete strengths based on historical lab 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.

Areas where historical lab test data may be used include (1) auxiliary building members at or above elevation 1007 feet and (2) containment internal structure (CIS) with the exception of concrete members adjacent to the reactor vessel or the floor of the reactor cavity.

USAR-5.11

Structures

Structures Other Than Containment

Rev 13

Safety Classification: **Safety**

Usage Level: **Information**

Change No.:	
Reason for Change:	
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Fort Calhoun Station

Table of Contents

List of Figures

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

5.11 Structures Other Than Containment

5.11.1 Classification of Structures

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

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. The AISC allows a 1/3 increase for allowable stress when the load combinations include wind or seismic load(s).

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. The capacity of Class I structural steel is determined in accordance with the allowable stress design provisions from the AISC Code, 1963 edition, and a 50% increase in allowable stress, but not exceeding 90% of the material yield strength (F_y).

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.

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 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.

5.11.3.5 Design Basis Load Combinations for Class I Concrete Structures

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

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

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

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

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

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

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

5.11.3.6 Special Case Load Combinations

a. Load Combinations for Faulted Conditions

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

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

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

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

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

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

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

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

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

b. Load Combinations for Spent Fuel Pool

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

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

where:

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

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

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

c. Special Conditions for Reactor Cavity and Compartment Walls and Transfer Canal

The reactor cavity and compartment walls and transfer canal are designed using ductility criteria at special locations following guidance of ACI 349-97, Appendix C – Special Provisions for Impulsive and Impactive Effects.

The structure can respond inelastically up to a ductility demand of $0.05/(\rho - \rho')$ not to exceed 10 in regions where moment governs and compartment pressurization is not present, and up to a ductility demand of 3 in regions with compartment pressurization loading where moment governs the design. Ratios ρ and $\rho`$ are defined in ACI 349-97.

For beams, walls, and slabs where shear or diagonal tension governs the design, the permissible ductility ratio shall be taken as:

(a) For shear carried by concrete alone, the permissible ductility ratio shall be 1.3;
(b) For shear carried by concrete and stirrups or bent bars, the permissible ductility ratio shall be 1.6;
(c) For shear carried completely by stirrups, the permissible ductility ratio shall be 3.0.

In addition, the requirements of ACI 349-97 Appendix C shall be applied.

Note that the ductility limits above are the same as those specified in ACI

349-97 Sections C3.3 through C3.9. Dynamic strength increase factors (DIF) shall be as specified in ACI 349-97 C.2.1. The other provisions of ACI 349-97 Appendix C are applied to ASTM A15 rebar similarly as permitted for ASTM A615 rebar and serve to insure that the structure has adequate ductility and reserve strength to withstand the imposed ductility demand.

The reactor cavity and compartment walls and transfer canal are designed using historical test data for concrete and steel reinforcement as designated in Section 5.11.3.8 and 5.11.3.9 respectively.

d. Special Condition for Containment Internal Structure

The containment internal structure beams and slabs [at elevation 1013'], outside the reactor cavity and compartment walls, are designed for an inelastic energy absorption factor of 1.25 for the vertical Maximum Hypothetical Earthquake loads through the use of a ductility modified response spectrum. The inelastic energy absorption factor accounts for the energy absorption capability of the structural components based on their ductility capability and is applied by direct reduction of the linear elastic derived applicable seismic response spectrum. The 1.25 factor results in effective limited ductility levels that are lower than values substantiated by ACI 349-97 and numerous reinforced concrete testing programs. This factor is not used for compression and shear loads in columns.

5.11.3.7 Codes and Standards

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

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

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

Reinforced concrete was designed in accordance with the Building Code Requirements for Reinforced Concrete, of the ACI (ACI-318-63) and as stipulated in Section 5.11.3~~.1a~~.

5.11.3.8 Concrete Compressive Strength

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:	Usage Level:
Safety	**Information**

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

Fort Calhoun Station

Table of Contents

List of Tables

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 to be based on testing in accordance with ASTM Standards. Historical lab 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. The yield strength for Grade 40 reinforcing steel may be determined from Certified Material Test Reports (CMTR), i.e., historical lab test data, in accordance with applicable ASTM standards. 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 may also be 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

Historical lab test data for several Class I Structures indicate that 28-day compressive strength test results exceed the material specification values used in the original design.

Where applicable, concrete design compressive strengths (f_c`) may be established using analysis of historical lab test data based on the minimum compressive strength test result of all samples environmentally controlled in the lab. This approach is in alignment with, but more conservative than the methodology of ACI 318-63, Chapter 5. Concrete strengths based on historical lab 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.

Areas where historical lab test data may be used include (1) auxiliary building members at or above elevation 1007 feet and (2) containment internal structure (CIS) with the exception of concrete members adjacent to the reactor vessel or the floor of the reactor cavity.

USAR-5.11

Structures

Structures Other Than Containment

Rev 13

Safety Classification:	Usage Level:
Safety	Information

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

Fort Calhoun Station

Table of Contents

List of Figures

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

5.11 Structures Other Than Containment

5.11.1 Classification of Structures

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

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. The AISC allows a 1/3 increase for allowable stress when the load combinations include wind or seismic load(s).

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. The capacity of Class I structural steel is determined in accordance with the allowable stress design provisions from the AISC Code, 1963 edition, and a 50% increase in allowable stress, but not exceeding 90% of the material yield strength (F_y).

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.

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 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.

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

5.11.3.6 Special Case Load Combinations

a. Load Combinations for Faulted Conditions

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

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

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

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

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

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

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

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

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

b. Load Combinations for Spent Fuel Pool

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

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

where:

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

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

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

c. Special Conditions for Reactor Cavity and Compartment Walls and Transfer Canal

The reactor cavity and compartment walls and transfer canal are designed using ductility criteria at special locations following guidance of ACI 349-97, Appendix C – Special Provisions for Impulsive and Impactive Effects.

The structure can respond inelastically up to a ductility demand of $0.05/(\rho - \rho')$ not to exceed 10 in regions where moment governs and compartment pressurization is not present, and up to a ductility demand of 3 in regions with compartment pressurization loading where moment governs the design. Ratios ρ and ρ` are defined in ACI 349-97.

For beams, walls, and slabs where shear or diagonal tension governs the design, the permissible ductility ratio shall be taken as:

(a) For shear carried by concrete alone, the permissible ductility ratio shall be 1.3;
(b) For shear carried by concrete and stirrups or bent bars, the permissible ductility ratio shall be 1.6;
(c) For shear carried completely by stirrups, the permissible ductility ratio shall be 3.0.

In addition, the requirements of ACI 349-97 Appendix C shall be applied.

Note that the ductility limits above are the same as those specified in ACI 349-97 Sections C3.3 through C3.9. Dynamic strength increase factors

(DIF) shall be as specified in ACI 349-97 C.2.1. The other provisions of ACI 349-97 Appendix C are applied to ASTM A15 rebar similarly as permitted for ASTM A615 rebar and serve to insure that the structure has adequate ductility and reserve strength to withstand the imposed ductility demand.

The reactor cavity and compartment walls and transfer canal are designed using historical test data for concrete and steel reinforcement as designated in Section 5.11.3.8 and 5.11.3.9 respectively.

d. Special Condition for Containment Internal Structure

The containment internal structure beams and slabs [at elevation 1013'], outside the reactor cavity and compartment walls, are designed for an inelastic energy absorption factor of 1.25 for the vertical Maximum Hypothetical Earthquake loads through the use of a ductility modified response spectrum. The inelastic energy absorption factor accounts for the energy absorption capability of the structural components based on their ductility capability and is applied by direct reduction of the linear elastic derived applicable seismic response spectrum. The 1.25 factor results in effective limited ductility levels that are lower than values substantiated by ACI 349-97 and numerous reinforced concrete testing programs. This factor is not used for compression and shear loads in columns.

5.11.3.7 Codes and Standards

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

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

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

Reinforced concrete was designed in accordance with the Building Code Requirements for Reinforced Concrete, of the ACI (ACI-318-63) and as stipulated in Section 5.11.3.

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.