UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE COMMISSION

In the Matter of:	)	
	)	
OMAHA PUBLIC POWER DISTRICT	)	Docket No. 50-285
	)	
(Fort Calhoun Station)	)	

OPPD RESPONSE TO THE SIERRA CLUB REQUEST FOR HEARING AND PETITION TO INTERVENE

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INTRODUCTION

Pursuant to 10 C.F.R. § 2.309(i), Omaha Public Power District ("OPPD") files this response to the request for hearing and petition to intervene filed on April 25, 2014, by the Sierra Club.[1] The Commission should summarily deny the Petition. The Petition seeks to circumvent the NRC's longstanding regulatory framework and fabricate a hearing opportunity where none exists. The Petition purports to be based on activities surrounding the process for restart of Fort Calhoun Station, but only offers vague assertions regarding modifications that are "necessary" or will "require" license amendments. The Petition does not reference any pending license amendment request, any specific notice of opportunity to request a hearing, or any expanded operating authority. Simply put, the Sierra Club cites no proceeding in which it could intervene.

1 "Petition to Intervene and Request for Adjudicatory Hearing by Sierra Club," dated April 23, 2014 ("Petition"). While dated April 23, 2014, the Petition was not actually filed or served on the parties until April 25, 2014.

BACKGROUND

A. Flood Protection Issues at Fort Calhoun Station

In a letter to OPPD dated October 6, 2010, the NRC issued a final significance determination for a Yellow finding identified in an inspection report dated July 15, 2010.[2] The NRC found that OPPD had failed to maintain written procedures for combating a significant external flood and that the site's procedures did not adequately prescribe steps to mitigate external flood conditions in the Auxiliary Building and the Intake Building up to 1,014 feet mean sea level ("msl"), as documented in the Updated Safety Analysis Report ("USAR").

In another inspection report dated May 11, 2012, the NRC found that: (1) OPPD's procedural guidance was inadequate to mitigate the consequences of external flooding; (2) OPPD failed to classify the six Intake Building exterior sluice gates and their motor operators as Safety Class III; and (3) Fort Calhoun Station did not meet design basis requirements for protection of the safety-related raw water system for flood levels between 1,010-1,014 feet msl.[3] The NRC determined that these violations were related to the previously issued Yellow finding.

B. Confirmatory Action Letter and NRC Restart Review

In April 2011, OPPD shut down Fort Calhoun Station for a scheduled refueling outage. In June 2011, a Missouri River flooding event began. On September 2, 2011, the NRC issued a Confirmatory Action Letter ("CAL") to OPPD, which documented actions that OPPD

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2 EA-10-084, "Final Significance Determination for a Yellow Finding and Notice of Violation, NRC Inspection Report 05000285/2010007, Fort Calhoun Station," dated October 6, 2010 (ML102800342).

3 "Fort Calhoun – NRC Integrated Inspection Report Number 05000285/2012002," dated May 11, 2012 (ML12132A395).

had committed to take prior to restarting the plant.[4] These commitments addressed impacts of flooding and other aspects of Fort Calhoun Station operations.

In a letter dated December 13, 2011, the NRC notified OPPD that it had made a change in the regulatory oversight of Fort Calhoun Station, transitioning NRC inspection and oversight from Inspection Manual Chapter ("IMC") 0305, "Operating Reactor Assessment Program," to IMC 0350, "Oversight of Reactor Facilities in a Shutdown Condition due to Significant Performance and/or Operational Concerns."[5] Under the IMC 0350 process, OPPD analyzed the extent of condition and the cause of known performance deficiencies. OPPD developed an Integrated Performance Improvement Plan ("IPIP") identifying actions to resolve the performance issues at Fort Calhoun Station. The comprehensive recovery effort involved numerous specific activities related to problem discovery and resolution, performance improvement, restart readiness, and regulatory margin recovery. OPPD also engaged a third-party to conduct a geotechnical and structural assessment of the post-flood condition and functionality of Fort Calhoun Station.

On June 11, 2012, the NRC issued a revised CAL ("Restart CAL") that included a Restart Checklist.[6] The revised CAL and Restart Checklist incorporated OPPD's IPIP commitments. On February 26, 2013, the NRC updated the CAL and associated Restart Checklist to include additional actions that OPPD had committed to take as part of the recovery effort.

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4 CAL 4-11-003, "Confirmatory Action Letter – Fort Calhoun Station," dated September 2, 2011 (ML112490164).

5 "Notification of Change to Regulatory Oversight of Fort Calhoun Station," dated December 13, 2011 (ML113470721).

6 CAL 4-12-002, "Confirmatory Action Letter – Fort Calhoun Station," dated June 11, 2012 (ML12163A287).

On December 2, 2013, OPPD submitted an "Integrated Report to Support Restart of Fort Calhoun Station and Post-Restart Commitments for Sustained Improvement" (ML13336A785) ("Integrated Restart Report"). This report detailed the actions OPPD took to address the Restart CAL, including: (1) the results achieved from implementing the IPIP; (2) the basis for closing the Restart Checklist items; (3) the completion of Flood Recovery Plan commitments; and (4) the actions taken to close the Restart CAL. Based on the NRC's review of OPPD's actions, the NRC determined that OPPD satisfied the commitments in the Restart CAL and the Restart Checklist.[7] OPPD subsequently restarted Fort Calhoun Station, reaching full power on December 26, 2013.

DISCUSSION

A. The Petition Should Be Summarily Dismissed

Section 189.a of the Atomic Energy Act mandates an opportunity for hearing on (among other things) license amendments. Here, the Petition seeks a hearing on four issues, but does not identify any pending or proposed license amendment or other licensing action that would give rise to a hearing opportunity.[8] In the absence of any proceeding in which to intervene or a licensing action on which to request a hearing, the Petition should be summarily dismissed.[9]

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7 EA-13-020, "Fort Calhoun Station Closure of Confirmatory Action Letter," dated December 17, 2013 (ML13351A423).

8 *See* Petition at 5-6 (acknowledging that "there is no Federal Register notice or any other type of notice as described in 10 C.F.R. § 2.309(b)(3) and (4)").

9 *See State of New Jersey* (Department of Law and Public Safety's Requests Dated October 8, 1993), CLI-93-25, 38 NRC 289, 292 (1993) (explaining that intervention is not available where there is no pending "proceeding" of the sort specified in Section 189.a).

The issues that the Petition seeks to address are related, in differing degrees, to actions that have been or may be taken by OPPD in response to the Yellow finding and the IMC 0350 restart process. But, Section 189.a does not require a hearing opportunity for every alleged non-compliance or performance deficiency. Assuming a non-compliance exists, licensees are obligated to take corrective actions in accordance with guiding principles such as 10 C.F.R. Part 50, Appendix B, Criterion XVI, as well as plant Technical Specifications and NRC Inspection Manual Part 9900 guidance on Technical Specification operability issues. Corrective actions that involve licensing basis changes are evaluated under 10 C.F.R. § 50.59 to determine whether a license amendment (and therefore a hearing opportunity) is necessary. Corrective actions that restore compliance with the licensing basis do not increase the licensee's operating authority, require an amendment, or trigger a hearing opportunity.

There is also no right to a hearing on NRC enforcement actions. All challenges to the adequacy of NRC oversight, including the adequacy of specific Section 50.59 evaluations performed by licensees, must be brought under 10 C.F.R. § 2.206.[10] In fact, the Sierra Club has already filed a Section 2.206 petition addressing many of the same issues raised in the Petition.[11] The Petition, and the NRC hearing process, simply are not the appropriate vehicles for disputing OPPD's corrective actions or the sufficiency of NRC enforcement activities.

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10 In *Southern California Edison Co.* (San Onofre Nuclear Generating Station, Units 2 and 3), CLI-13-09, __ NRC __ (December 5, 2013) (slip op. at 3-4, n.10), the Commission reiterated that Section 2.206 is the appropriate method to challenge licensee changes made under Section 50.59. And, contrary to the Sierra Club's claims (at 10) that the Section 2.206 process does not provide "a meaningful vehicle for the public to ensure that Fort Calhoun will be operated safely," the Commission reaffirmed the Section 2.206 petition as a viable method for obtaining relief. CLI-13-09, at 3-4.

11 "10 CFR 2.206 Petition Requesting the Nuclear Regulatory Commission to Revoke Omaha Public Power District's License to Operate the Fort Calhoun Nuclear Power Station," dated June 21, 2012 (ML12180A124). The Section 2.206 petition remains subject to NRC Staff review.

To the extent the Petition seeks a hearing on plant or procedure modifications (or licensing basis changes) that are still under development or that may be necessary in the future, the Petition is premature. In evaluating a plant or procedure change, OPPD will determine whether a license amendment is necessary by applying the Section 50.59 process at the appropriate time. If a license amendment is required, then there will be an opportunity for the Sierra Club and others to request a hearing. If no amendment is required, then OPPD may implement the change without prior NRC approval and without a hearing opportunity. The Sierra Club cannot seek a hearing in anticipation of a potential and undefined license amendment request that may or may not be submitted in the future. Hearing opportunities cannot be founded in speculation or inchoate plans of a licensee.

Because the Sierra Club failed to identify a pending license amendment or specific hearing opportunity, the Petition should be summarily dismissed.[12] The NRC's existing processes assure safety, including through review and approval of license and licensing basis changes where appropriate. Those processes also provide adequate opportunities for stakeholder participation, including through public meetings and a Section 2.206 petition. A hearing cannot be granted based on nothing more than a petitioner's desire for an additional forum in which to air concerns or complaints.

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12 The Petitioner also fails to demonstrate standing. Even assuming a license amendment is necessary, a petitioner cannot base his or her standing simply upon a residence near the plant unless the proposed action obviously entails an *increased* potential for offsite consequences. A petitioner must provide some plausible chain of causation to suggest that a license amendment would result in a distinct new harm or threat. *Commonwealth Edison Co.* (Zion Nuclear Power Station, Units 1 & 2), CLI-99-4, 49 NRC 185, 191 (1999). The declaration accompanying the Petition does not identify an increased risk of offsite radiological release or any other new harm.

B. Petitioner Has Not Identified Admissible Contentions Under 10 C.F.R. § 2.309

1. Contention 1: Flooding Modifications

In Contention 1, the Sierra Club asserts (at 16) that "[m]odifications for flood protection, including for protection of severe flooding in the event of upstream dam failures, require a license amendment." The Petition alleges that OPPD made significant modifications to flood protection equipment and procedures to address a design basis flood and also to protect Fort Calhoun Station from upstream dam failures.[13] According to the Sierra Club, these modifications require a license amendment and therefore entitle it to a hearing. The assertion is not correct.

a. Design Basis Flood Protection

Corrective actions to address non-compliances or the Yellow finding on flooding issues do not automatically require a license amendment. Here, however, OPPD previously identified two modifications that required a license amendment in accordance with the Section 50.59 process. As a result, there were two license amendment requests — and two hearing opportunities — associated with flooding corrective actions and modifications.[14]

The first license amendment request involved a Technical Specification change to revise the river level Limiting Condition for Operation and Surveillance Requirement in plant Technical Specifications, as well as the emergency action level entry condition.[15] The change,

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13 Petition at 16.

14 Beyond the two specific license amendments identified by OPPD, any challenge to the sufficiency of the NRC's enforcement response, including the scope of OPPD's corrective actions, must be made in a Section 2.206 petition.

15 LIC-12-0056, "Proposed Change to Revise Operating Requirements for Technical Specification 2.16, River Level, and Establish Emergency Action Level Classification Criteria for External Flooding Events under the Radiological Emergency Response Plan for Fort Calhoun Station," dated April 27, 2012 (ML12121A565).

which did not alter the physical design of the intake structure or any other plant structure, system or component, was the subject of a hearing opportunity in 2012.[16] The Sierra Club did not file a timely petition to intervene or request a hearing. The license amendment was subsequently issued by the NRC on January 28, 2014.[17] A petition to intervene now with respect to this past amendment is simply too late.

The second license amendment request would revise the design basis method in the site USAR for controlling the raw water intake cell level during periods of elevated river levels.[18] As the Sierra Club noted (at 18), these changes were discussed with the NRC during a public meeting held on April 22, 2013. An opportunity to request a hearing was published on March 18, 2014, with a deadline for requesting a hearing of May 19, 2014.[19] The Sierra Club did not request a hearing on this license amendment prior to the deadline. A petition to intervene based on this license amendment also is now untimely.

In any event, nothing in the Petition specifically challenges the adequacy of either the amendment already granted or the pending proposed change. The Sierra Club does not

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16 *See* 77 Fed. Reg. at 76082 (December 26, 2012) (discussing proposed amendment and providing notice of opportunity to request a hearing).

17 "Issuance of Amendment Re: Revision to Technical Specifications 2.16, 'River Level,' and 3.2, 'Equipment And Sampling Tests,' and Establishment of the Emergency Action Level Classification Criteria for External Flooding Events Under the Radiological Emergency Response Plan (TAC No. ME8550)" (ML14003A003).

18 OPPD, LIC-13-0105, "License Amendment Request (LAR) 13-03, Request to Revise Updated Safety Analysis Report to Allow Implementation of Modification EC 55394, *Raw Water Pump Operation and Safety Classification of Components during a Flood*," dated August 16, 2013 (ML13231A178).

19 *See* 79 Fed. Reg. 15144 (March 18, 2014) (discussing proposed amendment to revise the design basis method for controlling the raw water intake cell level during periods of elevated river levels and announcing an opportunity to request a hearing). The NRC Staff has not yet issued the license amendment.

present any information or expert testimony to dispute the adequacy of OPPD's amendment requests or any other flood-related plant modifications. The Sierra Club identified no specific deficiencies with OPPD's flood protection strategy. And, the Petition provides no expert testimony or factual information to question the adequacy of the flood protection strategy. As a result, there is no genuine dispute to litigate as required under Section 2.309, and therefore no admissible contention.

b. Upstream Dam Failures

The Sierra Club also asserts in the Petition that license amendments are necessary to address potential upstream dam failures. This claim is premature. The Petition cites a discussion between the NRC and OPPD on strategies to mitigate the effects of beyond-design-basis flooding held on April 22, 2013, but the Petition does not identify any basis for requiring a license amendment to address the potential for upstream dam failure at this time. Measures to mitigate a beyond-design-basis flood would not require an amendment — any enhancements would *increase* safety.

In response to the Fukushima Task Force recommendations, OPPD is currently performing a flooding hazard reanalysis for Fort Calhoun Station that will consider the potential for upstream dam failures. If necessary, an integrated assessment will then be prepared to evaluate the capability of flood protection systems to meet their intended safety functions under the reevaluated hazard. It is premature for the Petitioner to assert that modifications requiring a license amendment are necessary now or that a hearing opportunity is somehow ripe. Issues for ongoing regulatory consideration do not create current hearing rights. Instead, as the Petitioner implicitly recognizes through its prior actions, a Section 2.206 petition is the appropriate process for raising concerns that the NRC has not taken sufficient action to ensure safety.

The Petition also fails to demonstrate a genuine dispute with respect to the adequacy of OPPD's current plans to mitigate the effects of upstream dam failures and beyond-design-basis flooding. An NRC Backfit Appeal Panel specifically considered the potential for upstream dam failures to adversely impact Fort Calhoun Station and decided that there is not an immediate safety issue.[20] Nothing in the Petition specifically challenges OPPD's plans or the NRC Staff's conclusions regarding the need for immediate action. The Petition therefore fails to present a genuine dispute that could be the basis for an admissible contention under Section 2.309.

2. Contention 2: Design Basis Reconstitution Project

In December 2013, OPPD committed to performing a risk-focused reconstitution of the design basis, licensing basis, and USAR for Fort Calhoun Station. This effort includes a pilot project, to be completed in 2014, to reconstitute the design basis for one system. The commitment states that the overall project will be completed by the end of the fourth quarter 2018.[21] The NRC confirmed this commitment in a December 17, 2013 CAL, and this commitment remains open.[22]

The Sierra Club's proposed Contention 2 argues (at 25) that "[r]econstituting the design basis and licensing basis documents requires a license amendment." The Petition alleges that OPPD must obtain a license amendment to satisfy its "duty to update and maintain accurate

20 Memorandum to E. Collins from T. Blount, "Backfit Panel Regarding Fort Calhoun Flooding," dated March 6, 2012 (ML12229A184). The panel also noted that the issue is being addressed in OPPD's efforts underway in response to the Fukushima Task Force recommendations.

21 Integrated Restart Report, Encl. 3 at 12.

22 EA-13-243, "Confirmatory Action Letter – Fort Calhoun Station," dated December 17, 2013 (ML13351A395).

design basis documents."[23] No legal basis is provided for this assertion. The only factual support for the contention is a December 31, 2012 NRC inspection report, which included a non-cited violation for failure to maintain design basis documents.[24]

As explained above, corrective actions do not necessarily require a license amendment. In the case of Contention 2, OPPD's design basis reconstitution project involves reconciling design basis documentation — an administrative exercise that, in itself, does not inevitably result in any change to the plant, procedures, license, or licensing basis. Current operation is, and must be, in accordance with the license and Technical Specifications (including equipment operability determinations) no matter the status of the design basis reconstitution effort.

Additionally, the design basis reconstitution is ongoing and any assertion of a need for a license amendment is premature. In the process of reconstituting the design basis for each system, OPPD may in the future identify the need for additional engineering calculations or for physical changes to the plant, changes to procedures, or changes to the USAR. Any proposed changes would be subject to the Section 50.59 process. If a license amendment is necessary, then there will be an opportunity to request a hearing at that time. But, the Sierra Club cannot demand a hearing now on a change that has not yet been identified or even found to be necessary. There is no change to litigate.

Finally, the proposed contention is simply too vague to warrant a hearing. The Petitioner cites only an NRC inspection report, and challenges no particular aspect of the design basis reconstitution project. The Petition does not cite any past or pending change to a system

[23] Petition at 26.

[24] "Fort Calhoun – NRC Integrated Inspection Report Number 05000285/2012011," dated December 31, 2012 (ML12366A158).

that allegedly required a license amendment, nor does it identify any specific license change that is alleged to be necessary going forward. The Sierra Club presents no expert opinion or factual information to support its claim that the design basis reconstitution effort warrants a license amendment.[25] Contention 2 fails to establish a genuine dispute under Section 2.309.

3. Contention 3: Containment Internal Structures

In proposed Contention 3, the Petition claims (at 31) that possible future changes to repair or replace certain structural beams and columns require a license amendment. The Petition alleges that Fort Calhoun Station should not have been permitted to restart before these modifications were made and a license amendment granted. This contention is not based on any current hearing opportunity, is premature and vague, and to the extent the Petition cites factual information to support the contention, it is simply inaccurate.

Contention 3 references OPPD-identified discrepancies between design calculations and drawings for concrete beams at the 1049' elevation. In May 2012, OPPD discovered that certain Containment Internal Structure ("CIS") beams exceeded allowable loading conditions for both "working stress" and "no loss of function" as set forth in USAR Section 5.11, "Structures other than Containment."[26] In December 2012, OPPD discussed the CIS issue at an NRC public meeting, based on its preliminary assessment at that time. Later, in the Integrated Restart Report, OPPD committed to: (1) evaluate the structural design margin for CIS, reactor cavity, and compartments, and resolve any deficiencies to restore full structural design margin, (2) restore full structural design margin of Beams 22A and 22B, prior to

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25 Nor does the Petitioner justify the timeliness of filing this contention now, almost 18 months after the cited inspection report and several months after the CAL.

26 OPPD Presentation, "Fort Calhoun Station Containment Internal Structures (CIS)," dated December 12, 2012 (ML12349A151).

resuming operations following the first refueling outage after restart, and (3) evaluate the structural design margin and resolve any deficiencies to restore full structural design margin for the reactor head stand prior to its next use.[27] The NRC confirmed this commitment in the December 17, 2013 CAL, which states that OPPD will resolve any deficiencies in these areas in accordance with its Corrective Action Program.[28]

Citing slides presented by OPPD over a year ago at the December 2012 public meeting with the NRC, the Petitioner alleges "significant challenges" to using cast-in-place concrete for the CIS design modifications.[29] But, as clearly noted in the slides, the discussion reflected only the *preliminary* results of CIS design evaluations. The final design is still under development. In any event, evidence of ongoing evaluations or the need for corrective actions does not create a hearing opportunity. The Sierra Club cannot seek a hearing on hypothetical future modifications. Any modifications deemed necessary would be subject to the Section 50.59 process to determine whether the change requires a license amendment. If so, a hearing opportunity would be available at that time.

In the meantime, restart and operation of Fort Calhoun Station remains within the terms of the operating license and Technical Specifications. Safety equipment must be operable as required in the Technical Specifications. While the Petition alleges (at 33) that CIS members are not operable during normal operation, the contention is supported only by the preliminary

27 Integrated Restart Report at Encl. 2.

28 EA-13-243, "Confirmatory Action Letter – Fort Calhoun Station," dated December 17, 2013 (ML13351A395).

29 Petition at 33; OPPD, "Fort Calhoun Station Containment Internal Structures (CIS)," dated December 12, 2012 (ML12349A151).

(and now outdated) information presented by OPPD at the December 2012 public meeting.[30] OPPD subsequently finalized its operability assessment and determined that all CIS members were operable during both outage and normal operating conditions because the "no loss of function" requirement was satisfied. The NRC reviewed the calculations used to demonstrate CIS operability and found them acceptable.[31] To the extent that the Petitioner alleges current safety or compliance issues, or challenges an operability determination, Section 2.206 is the appropriate vehicle for seeking additional action, not the licensing hearing process.[32]

Lastly, the claims regarding CIS adequacy are vague at best. The Petition fails to challenge a specific issue with CIS operability or reference any particular change purportedly requiring a license amendment. The Sierra Club cites no expert opinion or factual information to support its claim that resolution of CIS issues warrants a license amendment.[33] For these reasons, Contention 3 does not establish a genuine dispute under Section 2.309.

4. Contention 4: Karst Geology

In proposed Contention 4, the Sierra Club argues (at 34) that "[m]odifications necessary to address the problem that the Fort Calhoun reactor was built above karst terrain require a license amendment." The Petition states that a geotechnical study at Fort Calhoun

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30 OPPD Presentation, "Fort Calhoun Station Containment Internal Structures (CIS)," dated December 12, 2012 (ML12349A151).

31 "Final Response to Task Interface Agreement 2013-05, Containment Internal Structures Operability Calculations at Fort Calhoun Station," dated January 28, 2014 (ML14016A260).

32 *See Vermont Yankee Nuclear Power Corp.* (Vermont Yankee Nuclear Power Station), CLI-00-20, 52 NRC 151, 169 n.14 (2000).

33 The Sierra Club also fails to establish the timeliness of the contention, which was filed 16 months after the OPPD presentation cited in the Petition, and several months after the NRC's CAL confirmed OPPD's commitments to resolve the CIS issues.

conducted after the June 2011 flood made reference to a 1968 study that notes the presence of karst formations at the site. According to the Petition, "there has been no serious effort to determine the nature and extent of the problem, and thus, no effort to address the problem."[34] The Sierra Club claims that addressing this issue will require major modifications and a corresponding license amendment.

The contention is premature and out of process — there are no pending or proposed modifications that are the subject of a current license amendment. The Sierra Club claims only that this issue "will require" a license amendment.[35] No legal basis is provided for a claim that a hearing is necessary now. NRC's rules of procedure do not allow members of the public to request a hearing based solely on their belief that a licensee should be required to make plant modifications or seek an amendment. Instead, any challenge to the current licensing basis or operational safety, or to NRC's oversight, must be brought under Section 2.206.[36] Contention 4 should be rejected on this basis alone.

The Petitioner also fails to demonstrate a genuine dispute with respect to the effects of karst geology on safety at Fort Calhoun Station.[37] Contrary to the Petition's assertions, this is not a new issue — it already has been thoroughly addressed by both OPPD and the NRC. The implications of karst geology at the site were considered and addressed in the original design

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[34] Petition at 34-35.

[35] *Id*. at 35.

[36] *See Entergy Nuclear Operations, Inc.* (Indian Point, Units 2 and 3), LBP-08-13, 68 NRC 43, 73 (2008) (explaining that the proper vehicle to challenge the adequacy of the USAR would be a Section 2.206 petition).

[37] According to the U.S. Geological Survey, "karst" is a terrain with distinctive landforms and hydrology created from the dissolution of soluble rocks, principally limestone and dolomite. *See* U.S.G.S. Groundwater Information Site, What is Karst? (available at http://water.usgs.gov/ogw/karst/pages/whatiskarst).

of Fort Calhoun Station.[38] During initial construction, OPPD specifically implemented procedures to verify that sound bedrock supported the piling for each Class 1 Structure.[39] The Sierra Club has not raised any specific issues with the adequacy of these historical efforts.

More recently, in the IMC 0350 process, the site's underlying karst was considered as part of Restart Checklist Item 2.b.3, *Impact of Subsurface Water on Soils and Structures*. According to the Integrated Restart Report (at 39), OPPD engaged several engineering firms to assess the changes to the foundation soils supporting the structures caused by the 2011 flood and any direct impact of the floodwater on the structures. Their findings are documented in the Plant and Facility Geotechnical and Structural Assessment report. No modifications to address karst geology were found to be necessary.[40] The NRC agreed, concluding that OPPD adequately addressed the flooding impact of sub-surface water on soils and structures and closing Checklist Item 2.b.3 in EA-13-020, "Fort Calhoun Station Closure of Confirmatory Action Letter," dated December 17, 2013. Any challenge to that conclusion would need to be made in a Section 2.206 Petition.

Contention 4 is in any event vague and unsupported. The Petition does not identify the particular "problem" posed by karst terrain at Fort Calhoun Station (other than

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38 *See* Attachment A – FCS USAR-5.7, "Structures, Piling, Rev. 3," dated May 19, 2011.

39 *See id*. at 7 (Section 5.7.1.2). Open ended steel pipe piles were driven to bedrock, then a small boring was advanced inside the pile to 15 feet below the tip of pile. The depth of 15 feet was predicated on the site bedrock exploration which concluded that cavities, where present, were detected within the depth of 15 feet below the bedrock surface. If no cavity was encountered in the boring, the rock was deemed sound; if a cavity was encountered, the pile was further driven through the cavity to sound rock. These procedures ensured that each pile was properly founded on sound rock.

40 *See* Attachment B – Excerpt of *Fort Calhoun Station Flood Recovery Action Plan 4.1*, "Plant and Facility Geotechnical and Structural Assessment," dated September 18, 2012, Revision 3 at 2-4, 3-10, 3-15, and 3-22.

general speculation about structural stability and groundwater contamination) or the modifications that it alleges would require a license amendment. And, the only factual support provided is inapplicable to Fort Calhoun Station. The Petition (at 38) discusses the difficulties associated with installation of "open-end concrete filled pipe piles." But, the piles at Fort Calhoun are not concrete filled.[41] The discussion of collapse mechanisms related to karst (cited by the Petitioner at 39-41) also is irrelevant to the Fort Calhoun Station site, which generally has non-cohesive soil above the potential karst area.[42] The Petition (at 41) also states that OPPD must consider cavities below bedrock surface. But, as noted above, this issue was specifically considered, and resolved, by OPPD during initial plant licensing and construction. At bottom, there is no factual support for the contention and no genuine dispute under Section 2.309.

CONCLUSION

The Petition should be summarily dismissed. The Petition does not identify any specific license amendment or hearing opportunity on which the Sierra Club bases its petition to intervene. Moreover, the Petition does not establish any genuine dispute related to a safety or compliance issue that could be the basis for an admissible contention.

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41 Attachment A at 7 (Section 5.7.1.2, *Pile Installation Procedure*)

42 *See* Attachment B at 2-8 (describing the site soils as non-cohesive).

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Dated in Washington, D.C.
this 20th day of May 2014

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE COMMISSION

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CERTIFICATE OF SERVICE

I certify that copies of the "OPPD RESPONSE TO THE SIERRA CLUB REQUEST FOR HEARING AND PETITION TO INTERVENE" and "NOTICES OF APPEARANCE" have been served on this 20th day of May 2014 by Electronic Information Exchange.

Respectfully submitted,

/s/ signed electronically by
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Dated in Washington, D.C.
this 20th day of May 2014

ATTACHMENT A

USAR-5.7

Structures

Piling

Rev 3

Safety Classification: **Safety**

Usage Level: **Information**

Change No.:	EC 51265
Reason for Change:	The USAR title pages and sections are being reformatted into the current format showing the safety classification and usage level. Additional editorial changes (e.g., font, capitalization, title corrections, etc.) are being made for consistency.
Preparer:	M. Edwards
Issued:	05-19-11 3:00 pm

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List of Figures

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

5.7 Piling

5.7.1 General

In order to provide information for the design of an appropriate foundation system, several series of subsurface exploratory programs were performed at the plant site.

Initially, a program comprising of 6 exploratory borings dispersed over the site area was performed at the time of acquisition of the property for a general indication of the nature of the overburden and bedrock.

Later upon determination as to where the major plant structures would be generally located within the site, a subsurface exploratory program including 16 borings was performed as discussed in Appendix B.

Upon establishment of precise location and outline of the plant structures, a program of 12 additional borings within these limits was instituted for final and corroboratory information. During execution of this program, evidence of a cavity was detected beneath the bedrock surface under the southwest corner of the plant.

Additional borings were then performed to delineate the extent of this cavity. Since all previous explorations at the site revealed sound limestone, it was believed that the detected cavitation as defined by this explained exploration was confined to this limited area. As a result, the plant location was moved upstream a distance of 90 feet in an attempt to avoid the region of observed cavitation, and an intensive program of bedrock investigation was instituted at the revised location.

To ensure maximum probability of interception of any cavities which could be present, borings were performed at randomly distributed locations. A total of 73 borings were made during this final investigation at the adjusted plant location to define the extent and frequency of cavity formation. Additional cavities were encountered and from the information obtained it was possible to establish some conclusions regarding their presence and nature as discussed in Appendix C.

5.7.1.1 Subsurface Conditions

Bedrock occurs at depths ranging from 58 to 67 feet below ground surface in the plant area. The upper four to eight feet is a massive, gray thick-bedded medium to fine grained oolitic limestone. Below the oolitic is a light gray, thin to moderately thick bedded, very fine grained aphanitic limestone, which ranges from 19 to 21 feet in thickness.

The cavities were encountered at the base of the oolitic limestone, extending from a few inches to approximately 14 feet into the aphanitic limestone and were believed to have developed by enlargement of major vertical joints by solution. Cavities tend to develop in the aphanitic limestone since the oolitic is more pervious than the aphanitic. The flow concentration is channeled downward and laterally along vertical joints in the less pervious aphanitic. The enlargement probably initiates at the interface of the two limestones, where the flow is first encountered, and progresses along the joints within the aphanitic limestone. Erosion progresses along the joints and results in long linear shaped cavities. A cavity may expand where a softer or more easily eroded material is encountered within the limestone. With time, weathering and spalling of the oolitic cap rock also takes place, causing enlargement of the cavity into the oolitic limestone, and resulting in a variation in the thickness of cap rock, and simultaneously the downward flow of water also enlarges the vertical joints within the oolitic limestone.

With the disclosure of cavities in the bedrock beneath the plant site, and with the plant arrangement and elevation based on support of the plant on piles to bedrock, it was necessary to select a pile type and method of installation which would permit investigation of the bedrock at each pile and application of corrective measures where necessary, to ensure that each pile was properly founded on sound rock.

To comply with these requirements, open end steel pipe piles were selected, and installed in accordance with the procedure described in Section 5.7.1.2.

5.7.1.2 Pile Installation Procedure

Piles were driven open end onto bedrock. An exploratory small diameter probe was made within the pile from the bedrock surface extending to a depth of 15 feet to determine if any cavity was present under the pile. The depth of 15 feet was predicated on the basis of the findings from the site bedrock exploration which established that cavities, where present, were detected within the depth of 15 feet below the bedrock surface. If no cavity was encountered, in effect confirming the soundness of the bedrock, the pile installation was considered as complete.

To ensure that the piles were actually seated on rock, a check was made for every pile of its bottom elevation after driving refusal was reached against the elevation of sound rock as indicated by the exploratory probe. Piles that were found not to have reached rock were further driven to the proper seating level on the rock.

Where a cavity was revealed from the results of the exploratory probing, a determination was made of the depth beneath the initially encountered bedrock surface necessary for seating of the pile onto sound rock. The pile was cleaned out and by means of an under reaming expansion rock auger working from within the pile in its initially driven position a hole of slightly larger diameter than the pile was drilled beneath the base of the pile, extending to the predetermined depth to sound rock and providing a flat base for seating of the pile.

Prior to resumption of pile driving, its length was checked against that required to ensure reaching to the level of sound rock at the bottom of the under reamed hole. If it was found to be too short, the pile length was extended by welding on an additional section to the upper end as required.

All welding operations were performed in accordance with rigidly established and enforced procedures. Welds were of the full penetration type, using the manual metal arc process. A base metal preheat temperature of 250°F was applied.

Finally, upon assurance of the sufficiency of the pile length, pile driving was resumed for proper seating onto sound rock. Piles were considered as seated when movement under ten blows each equal to at least 55,000 foot pounds of energy, was no more than one quarter inch. In actual construction, the piles were driven with a pile driver rated at approximately 57,000 foot-pounds of energy per blow. Refusal criteria for the piles was retained at ten blows per one-quarter inch.

By means of this pile installation procedure it was possible to investigate the condition of the bedrock at each individual pile and to apply appropriate corrective measures where necessary to ensure that each pile was properly seated on sound rock.

Plans and elevations of Class "A" piling installations are shown on Figure 5.7-1 and Figure 5.7-2.

5.7.1.3 Arrangement at Tops of Piles

After completion of seating of the piles onto sound bedrock, the tops of piles were cut off to the proper elevation and ground as required to provide a true plane surface for seating of cap plates. The cap plates are of suitable size for transfer of loading from the base mat concrete to the pile. The piles are embedded into the foundation mat for a length of three feet.

Detail of piling-to-base mat connection is shown on Figure 5.7-1.

5.7.2 Pile Loading Tests

5.7.2.1 General

To confirm the appropriateness of the pile selected for supporting the containment and auxiliary building structures from the aspects of both feasibility of installation and load capacity, actual tests were performed on the various piles considered at the design stage.

After it became evident, by the encountering of cavities within the limestock bedrock, that conventional steel H piles were not appropriate, tests were conducted on the feasibility of utilizing steel pipe piles, driven open end and concrete filled. To this end, a number of such piles of potentially appropriate size and capacity were installed and investigated, under a pile testing program (See Appendix D).

It became obvious during the course of this program that no dependence could be placed on the concrete fill within the pipe pile for any contribution to the pile capacity, because of the inability to satisfactorily remove the interior soils without affecting the surrounding soils and to adequately install the concrete for direct positive bearing on the bedrock surface.

As a result, it was necessary to select a pipe pile size of adequate capacity based upon steel cross sectional area only, without reliance on a concrete core.

As a final check, after completion of the subsequent soils densification operation, all piles were retapped to the refusal criteria of ten blows per one-quarter inch to ensure proper seating.

To eliminate any possibility of liquefaction occurring under Class I structures during the maximum hypothetical accident, the soil beneath these structures was densified to a relative density that will preclude this possibility.

After installation of the piling for the reactor building, auxiliary building and intake structure the in-situ sands between the piles were densified by vibroflotation. The pattern of vibroflot insertions was coordinated with the piling, the maximum spacing between insertions being on the order of six feet with the average somewhat less. The densification was performed from the level of the underside of the foundation mat to the top of rock and covered the entire area of the reactor building, auxiliary building and intake structure. The criterion used was that average relative density should be not less than 85% and the minimum not less than 70%.

After densification, a total of 83 borings were drilled into the compacted material to evaluate the vibroflotation results. Standard penetration tests were performed at three feet vertical intervals in each boring and the relative density of the sand was determined in accordance with Gibbs and Holtz's correlation between relative density and spoon penetration resistance (Reference 5-13). If an individual boring indicated unsatisfactory results the extent of the unsatisfactory material was determined by drilling additional borings. All soils of unsatisfactory density were recompacted and additional borings were drilled to certify that adequate compaction was achieved. A statistical analysis based on 696 standard penetration test results indicates an overall confidence level of 96.6% that the average relative density for the entire area is not less than 85%.

5.7.2.2 Selection of Pile

The pile size thus selected, investigated, and ultimately utilized for the foundation piling was 20 in. O.D. with 1.031 in. wall thickness as manufactured under the requirements of the American Petroleum Institute Specifications for Line Pipe, designated API Std. 5L Grade B (35,000 psi minimum yield strength).

5.7.2.3 Testing Procedure

Two piles were installed at the bottom of a cofferdam constructed to the anticipated construction grade. The piles were then tested to evaluate their compressional, uplift and lateral capacities.

Each pile was subjected to a compression loading of 650 tons representing approximately twice the maximum vertical load design capacity of the pile section in accordance with the requirements of the AISI Pile Foundation, Fourth Edition, 1963.

One pile experienced a total vertical deformation at the top of the pile in the order of 3/4 inch and a net settlement after removal of the load of 1/4 inch. The other pile indicated a total gross deformation of slightly over an inch and a net settlement of less than 1/4 inch. One of the piles was instrumented with strain measuring apparatus (tell tales), results from which indicated that less than 10 percent of the compressive load was taken in skin friction and the remainder in end bearing on the limestock bedrock.

After completion of compressive loading tests, an uplift test was performed on each pile. The first pile experienced a yield resistance to pull out of approximately 55 tons. The uplift test on the second pile revealed a total resistance to upward force on the order of 65 tons. These capacities were consistent with results obtained from uplift tests performed earlier on smaller concrete filled pipe piles.

Lateral load tests were performed by development and application of horizontal load to each of the piles by hydraulic jacking between the two test piles. Due to physical limitations, this test did not attempt to duplicate the situation of the piles in actual construction, in which the embedment of the piles in the foundation mat creates a degree of fixity at the top of piles. It was recognized that this test simulating free head individual pile behavior would result in larger deflections per unit amount of applied load than for fixed headed piles.

However, the data derived therefrom were considered valid in confirming soil parameters developed during tests of piles performed earlier during the initial phase to the program, and pile displacements could be converted from free-ended to fixed-ended conditions.

The test of the free ended piles indicated lateral deflections at the tops of the piles of from four to six inches at a horizontal load of 120 tons.

5.7.2.4 Conclusions

The following pile design capacities and criteria were established on the basis of the data obtained from the pile loading tests:

a. Compression: Design capacity: 325 tons per pile. Corresponding maximum pile vertical deformation: one-quarter to one-half inch.

b. Uplift: Maximum ultimate uplift capacity has been assessed at 40 tons per pile. For design use this value was modified by a factor of safety appropriate to the nature of the application.

c. Lateral Load: Pile behavior was determined to be in accord with conventional lateral pile capacity theories up to the elastic limit of the pile-soil system. Beyond that point predictions regarding pile behavior were based on the data developed in the load test program.

Secant coefficients of horizontal subgrade reaction for use in foundation design are presented in Table 5.7-1. These coefficients were conservative in that they reflect data realized in the test program for subsurface conditions as then existing, whereas subsequently the entire soil block beneath the structure foundation was densified.

Table 5.7-1 - Secant Coefficients of Horizontal Subgrade Reaction

Allowable Deflection (inches)	Coefficient n_h* (lb/in^3)
1/4	40
1/2	33
3/4	27

* See Appendix D

5.7.3 Loading and Design Criteria

The piles for the foundation under the containment and auxiliary building structures were designed for the loading conditions and combinations previously outlined in Section 5.5.2 for the containment structure concrete shell. The determination of pile size, number and arrangement was made on the basis of the most conservative requirements obtained by comparison of the results of two independent methods of design. The following criteria and methods were utilized:

a. For the loading combinations given for working stress design, Section 5.5.2.2, the piles were designed in accordance with the basic formula,

$$\frac{fa}{Fa} + \frac{fb}{Fb} \leq 1$$

where: fa = computed axial stress
fb = computed bending stress
Fa = axial stress that would be permitted if axial stressalone existed, in accordance with AISI, Pile Foundations, Fourth Edition, 1963.
Fb = 60% of the specified minimum yield strength of the steel. When wind or design earthquake loadings (vertical and lateral) are included, the formula was modified to the following:

$$\frac{fa}{Fa} + \frac{fb}{Fb} \leq 1.33$$

b. For the factored load equations given for modified ultimate strength design, Section 5.5.2.3, and no loss of function design, Section 5.5.2.4 the maximum stresses permitted for the piles was the guaranteed minimum yield strength of the steel. The 0 factor is 0.90.

c. The soil reaction modulus (sometimes referred to as the coefficient of lateral subgrade reaction) was assumed to vary linearly with depth:

$K = K_0 + K_1 X$

where: K = soil reaction modulus, psi
X = depth

The following values of K_0 K_{11} have been used for design:

$K_0 = 0$;
$K_1 = n_h = 35$ lb/in^3 for design earthquake;
$K_1 = n_h = 17.5$ lb/in^3 for maximum hypothetical earthquake

d. The difference-equation method for elastic pile theory was used for the determination of pile stresses.

No reduction in vertical pile load capacity due to group action was considered since all piles were driven to essential refusal on bedrock.

All lateral loads were assumed as resisted directly by the piles, and then transmitted to the soil block through the piles.

The tops of the piles were assumed to be restrained against rotation for design and analysis purposes by their embedment into the foundation mat.

The pile section design properties were based on the assumption of a 1/16 inch reduction in wall thickness as an allowance for corrosion. This has been introduced as an additional conservatism since the piles are protected against corrosion by a cathodic protection system (see Section 5.7.5).

Pile design loadings are shown in Table 5.7-2.

Table 5.7-2 - Pile Design Loads

		Maximum Load per Pile (Kips)
Vertical Loading Summaries for the Following Combinations of Concurrent Design Conditions:		
I.	Dead Load + Live Load + Post-Tensioning + Operating Temperature	360
II.	I + Accident Design Pressure + Design Earthquake	580
III.	I + Accident Design Pressure + Maximum Hypothetical Earthquake	610
Horizontal Loads Due to Earthquake:		
Design Earthquake		44
Maximum Hypothetical Earthquake		68

5.7.4 Seismic Considerations

A related phase of work concerned investigation and application of appropriate measures to the soil beneath the plant foundations to ensure stability against liquefaction when subjected to seismic disturbance. Preliminary studies of the soil in its initial undisturbed state indicated that there was a potential tendency for liquefaction to occur, and established the need for further investigation and development of appropriate criteria as guidelines. The criteria subsequently established dictated that to ensure against liquefaction of the soils for the seismic intensities postulated relative densities of 85 percent average with a 70 percent minimum were required. Measurement of soil densities was made by means of standard penetration tests, and evaluation of observed blow counts were determined in accordance with data presented by Gibbs and Holtz (Reference 13).

Upon completion of the foundation piling installation, a check of the soil densities indicated that additional densification was necessary to meet the specified criteria. The Vibroflotation system was subsequently utilized to provide the necessary densification of the soil from the top of the bedrock to the underside of foundation to the specified values of relative densities.

5.7.5 Corrosion Protection

Although preliminary chemical analysis performed on the soils and ground water at the site indicated that the sub-surface material is only slightly basic and its effect on embedded steel material would be insignificant, subsequent soil-resistivity investigation revealed that the underground environment could be mildly corrosive to buried, unprotected steel. If no precautions were taken it is possible that some metal loss could occur. Therefore, to ensure the integrity of the piles, a system of active, electrolytic corrosion protection was provided, and as an additional precaution a 1/16 inch corrosion allowance was included in the pile wall thickness.

Cathodic Protection Service of Houston, Texas, was engaged as consultants to review accumulated data, make necessary further tests, and design a comprehensive impressed-current system for protection of all steel, but with particular emphasis on retaining the full, structural integrity of the pile system. The recommendations of that organization were followed in the design of plant and substation grounding systems to ensure compatibility with the corrosion protection system.

The number, size, and distribution of impressed-current anodes ensure the capability of supplying one milliampere, dc, to each square foot of surface of steel to be protected with no more than 50 percent anode weight loss in 40 years. To meet this requirement, a total of 416, 3-inch by 60-inch, high-silicon, cast-iron anodes were installed. Impressed-current anodes are buried in a surround of coke breeze.

Twenty-six zinc reference anodes were installed to permit periodic checks for system polarization and re-adjustment of anode-group currents to maintain proper operation. The containment liner, reinforcement, and tendon sheath steel are electrically interconnected to each other and to the piles.

The containment liner plate was coated on the exposed face with an application of 4 dry mils of Carboline Phenoline 305 over a 3 mil base coat of Carbozinc 11. The rear face of the liner plate is unpainted; concrete of the containment shell was poured directly against it and protects it against corrosion.

The tendon system was protected against corrosion after installation and stressing of the tendons by filling the tendon sheaths and caps with a corrosion preventative grease. The caps enclosing the end anchorage of the dome and wall tendons at the ring girder were protected by a corrugated aluminum siding enclosure.

Reinforcing steel of the reactor containment building, the reactor auxiliary building, and the mat were connected to the plant grounding system, the steel piles, and thus, if exposed to ground water are afforded the same cathodic protection as the piles.

ATTACHMENT B

Omaha Public Power District

Fort Calhoun Station
Flood Recovery Action Plan 4.1
Plant and Facility Geotechnical and Structural Assessment

September 18, 2012

Revision 3

Final Report

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Preface

As the discovery process progressed, the Plant and Facility Geotechnical and Structural Assessment Report (Assessment Report) was periodically updated as indicated below. The issuance of the initial revisions of the Assessment Report was intended to provide early documentation of results. It was understood that subsequent revisions would add new information that would increase confidence in the results and conclusions. It is important to note that each revision includes the information contained in previous revisions.

- Revision 0, Issued 10/14/2011 – First issuance of report to OPPD by HDR. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure.
- Revision 1, Issued 11/28/2011 – Incorporated results of geotechnical drilling program (including majority of data from subcontractors), geotechnical comparative analysis, and additional surveys and site monitoring. These data increased the confidence level in the conclusions for some structures.
- Revision 2, Issued 05/04/2012 – Incorporated results of the forensic investigations for Key Distress Indicators, early 2012 soil testing and investigation, and assessment of Priority 2 Structures.
- Revision 3, Issued 09/18/2012 – Incorporated responses to reviewer comments, along with additional soil testing and investigation (performed in mid-2012).

It should be noted that early revisions of the Assessment Report provided preliminary results and conclusions that may be clarified, revised, corrected, and/or confirmed in later sections of the Assessment Report. Therefore, all sections should be reviewed in their entirety before drawing any conclusions from this Assessment Report.

400-mile radius of FCS. This is consistent with USAR Section 2.4, which briefly discusses the structural geologic setting of the FCS site with respect to historical seismicity. USAR Section 2.4 also states that no faulting is evident in the Pleistocene and recent sediments of the Missouri River Lowlands and that known faults in the vicinity of the FCS site exhibit no evidence of movement in historic times.

2.1.3 Seismic Hazard

Assessment of seismic hazard is based on the earthquake characteristics and the causative fault associated with the earthquake. These characteristics include magnitude of maximum earthquake, distance from the site to the causative fault, fault length, and activity of the fault. The effects of site soil conditions and the mechanism of faulting are accounted for in the attenuation relationships.

The probabilistic strong ground-motion values were developed from USGS gridded databases, developed by Frankel, et al. (1996 and 2002), and with most recently developed Next Generation Attenuation (NGA) relationships by Petersen, et al. (2008). These values were queried from USGS-maintained databases located at http://gldims.cr.usgs.gov/website/nshmp2008/viewer.htm and https://geohazards.usgs.gov/deaggint/2008/. The results of this analysis are presented in Attachment 1, Deaggregation Plots. Attachment 1 illustrates the regional probabilistic strong ground motion for the 10 percent probability of exceedance in 50 years, 2 percent probability of exceedance in 50 years, 2 percent probability of exceedance in 100 years, and 2 percent probability of exceedance in 200 years. Estimated peak ground acceleration (PGA) is summarized in Table 2-2.

Table 2-2 – Peak Ground Acceleration as Percentage for Various Return Periods

Earthquake Return Period (years)	Approximate Probability of Exceedance in 50 years (%)	Peak Ground Acceleration[A]
500	10	0.0142 g
2500	2	0.0431 g
5000	1	0.0669 g
10,000	0.5	0.1020 g

[A] - Peak ground acceleration is measured by the acceleration due to gravity (g).

Source: USGS. July 21, 2011. “2008 Interactive Deaggregations (Beta).” *Geologic Hazards Science Center*. Retrieved September 20, 2011. https://geohazards.usgs.gov/deaggint/2008/.

The PGA values presented in Table 2-2 are based on USGS probabilistic seismic hazard analyses for various return periods and are useful for presenting an overall seismic hazard for a geographic area. These values are not for the purpose of establishing seismic design criteria such as the design earthquake (0.08 g) and maximum hypothetical earthquake (0.17 g) that are presented in USAR Section 2.4.3. The USAR values are based on a detailed deterministic seismic hazard analysis that uses site-specific and site-area-specific data to develop PGA values.

2.1.4 Site Geologic Hazards

Several geologic hazards have been identified at the FCS site and discussed in previous design reports by Dames & Moore (January 26, 1967, and January 30, 1968). These hazards include the existence of karst features associated with dissolution of the Winterset Member of the Dennis Formation

Limestone, liquefaction of the loose poorly graded sands identified at the site, bank slope stability adjacent to the Missouri River, and scour and erosion of near-surface soils.

2.1.4.1 Karst

Dames & Moore (January 30, 1968) identified at least two significant karst features in the Winterset Member of the Dennis Formation Limestone that apparently have developed along existing fractures. The features were estimated to be as much as 5 ft wide, 16 ft deep, and 45 ft long and consist of an upper 1.5- to 3-ft void and a lower zone of decomposed limestone and detritus. The approximate location of these features is shown in Figure 2-1, Geotechnical Areas and Cross-Section Locations. Cross-sectional views of the geologic setting are presented in Figures 2-2 and 2-3. Figure 2-2, Section A-A, shows where these karst features approximately intersect the subsurface section.

Pile installation at FCS for the Containment, Auxiliary Building, Turbine Building, and Intake Structure was designed to penetrate any overlying layer of limestone that covers the karst feature and to found the pile on sound rock at the bottom of these features. The potential influence of these karst features on foundation stability is considered minimal. It is likely that additional karst features exist across the site, but the overlying alluvial cover of a minimum of 61 ft offers a buffer to the influence of these features on any structure. Further dissolution of limestone is an assumed process given that the limestone is in contact with groundwater. The most aggressive dissolution of limestone by groundwater occurs in the vadose zone (Mylroie, 1984). The fact that the karst features at the FCS site are covered by approximately 60 ft of alluvial material and are in contact with groundwater that has experienced some subsurface residence time dictates that the rate of karst feature development (limestone dissolution) is low. In addition, the karst features encountered in the 1967 Dames & Moore drilling program were primarily filled with decomposed limestone and detritus. The volume of space needed to allow significant collapse of overlying soils is not present. Therefore, within the expected service life of FCS, the process of limestone dissolution is not significant.

A further understanding of the karst features at the FCS site would require drilling and installation of sampling wells to sample water near the limestone and soil contact in order to assess the chemical characteristics of the groundwater at this interface. This effort is not considered necessary as part of this Assessment Report because the plant has functioned without evidence of foundation subsidence due to karst feature collapse and resulting collapse of overlying soil prior to and during the 2011 flood.

2.1.4.1 Liquefaction of Non-Cohesive Soils

Liquefaction studies have been performed by others for the FCS site using post-construction conditions. The assumptions used in performing the liquefaction analyses and results of those studies have not been reviewed by HDR, but it is believed that the largely non-cohesive, saturated soil materials at the site would be subject to liquefaction given sufficient seismic loading.

2.1.4.2 Bank Slope Stability

The site has slopes along the Missouri River that could experience stability problems due to river-level increase and then rapid drawdown, resulting in excessive pore pressures in the slopes of the river bank that are adjacent to any of the FCS structures. The mostly non-cohesive nature of the soils likely allowed drainage and dissipation of pore pressure without significant effects on channel slopes.

2.1.4.3 Scour and Erosion

The inundation of the site has the potential to scour and erode the existing grade and remove soil material from around and beneath structures that are founded near the ground surface. The non-cohesive nature of the site soils indicates scour potential given sufficient water velocity and capacity to carry sediment.

2.2 Geomorphology and Physiographic Setting

FCS is located in northeastern Washington County, Nebraska, approximately 4 miles southeast of Blair, Nebraska. The site lies within the Central Lowland portion of the Interior Plains Physiographic Province, as shown in Figure 2-4 (USGS, 2003). More specifically, the site is classified as part of the Dissected Till Plains, a subdivision of the aforementioned province, a region covered by Pleistocene glacial events that deposited till during glacial advance as well as during glacial retreat. The till has since been partially covered with eolian (wind-deposited) loess deposits and dissected by erosion caused by the Missouri River and its tributaries.

Washington County is also recognized as having two distinct physiographic divisions: 1) uplands formed in loess and glacial till; and 2) floodplains along the Elkhorn and Missouri rivers (U.S. Department of Agriculture, Natural Resources Conservation Service [USDA NRCS], 2004). In addition, the floodplains of the Missouri River are subdivided into the low bottom, which consists of a frequently flooded zone of meander scars and oxbow cutoffs, and the flood basin, which lies between the low bottom and the uplands. The flood basin is less frequently flooded than the low bottom.

2.3.4.6 Debris Impact

Floodwater carries debris ranging from large branches and trees to storage tanks and mobile homes. Debris that impacts a structure imparts a load on the structure that depends on the weight of the debris object, the velocity of the floodwater, the location on the structure where impact occurs, and the duration of the impact.

2.4 Geotechnical Baseline

2.4.1 In-Situ Soil Characteristics

Dames & Moore conducted a site subsurface investigation in 1967. A total of 89 borings were drilled during this field investigation to assess the properties of the site soils and bedrock, as show in Figure 2-16. Dames & Moore published the results of their 1967 field work in a January 30, 1968, report titled "Foundation Studies, Fort Calhoun Station Number One, Near Fort Calhoun, Nebraska," in which they drew the following general conclusions regarding the subsurface soil characteristics:

- The surficial soils consist of loose fine sands with varying amounts of silt to approximately 10 ft.
- Depths from 10 ft to approximately 30 to 35 ft generally consist of loose to compact (dense) fine sand.
- A 5- to 10-ft layer of compact (dense) fine sand lies below the loose to dense fine sand.
- Below the dense layer is a less compact (dense) layer of poorly graded to well-graded sand with thin layers of silty clay and some gravel.

Based on laboratory-determined relative densities, the relative density of the subsurface soils ranged from 47 to 82 percent. The field investigation involved standard penetration tests (SPTs) and the recording of N values for the soils. The N value, reported in blows per foot, is the number of blows required to drive the sampler for the last 1 ft of the sampling interval. There is no indication as to whether the values are normalized N60 values (corrected to 60 percent of the theoretical energy delivered by an SPT safety hammer) or are uncorrected values, so the values are assumed to be uncorrected. In addition, a standard SPT sampler and the Dames & Moore Type U soil sampler were used to record N values, and a 300-pound hammer at a 24-in. fall and a 140-pound hammer at a 30-in. fall were used to impart the energy to drive the samplers. The net effect on N values is not documented. N values are depicted in Figure 2-2, Section A-A, and Figure 2-3, Section B-B.

These findings are generalized to represent overall site conditions, but localized variations are presented in Figures 2-2 and 2-3. The locations of the section lines and the approximate plan view location of the known karst features are presented in Figure 2-1, Geotechnical Areas and Cross-Section Locations.

Much of the upper 10 to 15 ft of in-situ material was actually logged as low-plasticity silt with varying amounts of sand. N values from this zone were generally lower than 10. The zone below this, described by Dames & Moore (January 30, 1968) as loose to dense fine sand 30 to 35 ft thick, is shown as poorly graded sand (SP) in Figures 2-2 and 2-3. This zone appears to be consistent across the FCS site; however, the zone of dense fine sand is not as consistent as the Dames & Moore report implies. N values in borings B-27 and B-108 range from 79 to 125 at depths ranging from 35 to 50 ft from existing (at the time of the exploration) ground surface, while borings B-29 and B-28 show N values of 14 to 48 for a comparable depth range less than 100 ft away from borings B-27 and B-108.

The zone of less dense, poorly graded to well-graded fine sand with varying amounts of silt and some gravel is generally consistent across the site and makes up the 15 to 20 ft of alluvial material on top of bedrock.

Limited laboratory testing was completed for soil samples and includes particle size analyses. Particle size analyses showed predominantly fine sands with minor fractions of silt and medium-grained sand.

2.4.2 Rock Mass Characteristics

According to the Dames & Moore (January 30, 1968), bedrock was encountered at depths ranging from 58 to 67 ft and varied from el. 931 to 935 ft. The rock encountered was identified as the Winterset Member of the Dennis Formation Limestone of the Pennsylvanian Kansas City Group. The bedrock at the site was described as having an upper zone 4 to 8 ft thick and consisting of massive, gray, thickly bedded, medium- to fine-grained oolitic limestone. Below this zone was a zone of light gray, thinly to moderately bedded, fine-grained limestone (referred to as aphanitic in the Dames & Moore report) having 0.5- to 2-in.-thick shale layers. Karst features were found in this lower "aphanitic" layer as briefly discussed in Section 2.1.4.1, Karst, above, but also included part of the overlying oolitic limestone as recorded in borings B-104 and B-104B. Figure 2-2, Section A-A, and Figure 2-3, Section B-B, present representative subsurface depth and thickness of the site bedrock. The locations of the section lines and the approximate plan view location of the known karst features are presented in Figure 2-1, Geotechnical Areas and Cross-Section Locations.

The rock mass was logged as "unweathered" ("fresh" using the U.S. Bureau of Reclamation *Engineering Geology Field Manual* [1998]) and hard, and rock quality designation (RQD) values ranged from 97 to 100 percent with few exceptions related to solution features (karst). Specific findings were as follows:

- A zone of moderately to intensely weathered limestone in boring B-116 was logged at the bottom of the oolitic limestone and 4 ft into the underlying fine-grained limestone, and an RQD value of 40 percent was recorded within this zone. This was a solution feature that had not yet, through chemical dissolution of the limestone, developed into a void and a zone of completely decomposed limestone.
- A large solution feature was intercepted by borings B-104, B-104A, and B-104B from depths of 63 to 79.2 ft (el. 932.3 to 916.2 ft) that had an upper 2 to 3 ft of void and the remaining lower portion filled with decomposed limestone.
- Borings B-72 through B-72H were drilled to define the extent of a large solution feature that ranged in depth from 65.6 to 77.7 ft (el. 932.1 to 920.0 ft).
- Borings B-30 through B-30Q were drilled to define the extent of a solution feature that ranged in depth from 67 to 83 ft (el. 929.7 to 913.7 ft).
- Borings B-103 and B-103A encountered a more limited but possibly connected zone of dissolution that ranged from el. 934.5 to 936 ft.
- A zone of increased weathering, RQD values ranging from 42 to 55 percent, and a 1.5-ft void were encountered in boring B-141 from depths of 70 to 77 ft (el. 926 to 919 ft).
- Boring B-108 drilled through a cavity from depths of 65.7 to 75.0 ft (el. 928.8 to 919.5 ft).

These noted solution features were recognized by Dames & Moore as following predominant fracture sets that were reportedly mapped at a local quarry. The orientation of these fracture sets is reportedly N50E and N58W.

The potential for the enlargement of solution features (karst) in the bedrock portion of the foundation to be a foundation failure mechanism due to flooding events is minimal. The pile design for the Containment, Auxiliary Building, Turbine Building, and Intake Structure called for pile installation past any weathered zone to the bottom of any known or encountered solution feature. In addition, the limestone bedrock is covered by a minimum of 61 ft of soil cover, so acidic atmospheric water is not likely to reach the karst features. The only plausible mechanisms for continued karst development are 1) a connection to the river bottom that allows chemically aggressive (acidic and not saturated with respect to calcium) water into a karst feature, and 2) a scenario in which the overlying soils do not alter the chemistry of the groundwater so that it maintains the potential to dissolve the limestone. These mechanisms take significant time relative to the operating life of the FCS structures and are not significantly related to a plausible failure mechanism.

2.4.3 Groundwater

Prior to construction, groundwater was described by Dames & Moore (January 26, 1967) as generally within 2 ft of the surface at the site and sloping gently to the east toward the Missouri River. Groundwater elevations and river elevations prior to the 2011 flood event and after the onset of the flood event are presented in Table 2-6. An increase in groundwater elevation on the order of 10 ft has been recorded as a result of the 2011 flood. The data do not include groundwater elevations at the peak flood elevation of 1006.85 ft because groundwater measurements were not recorded during peak flood levels. Groundwater and river elevations for December 10, 2010, and June 4, 2011, are shown in Figure 2-2, Section A-A, in order to present the general response of groundwater elevations relative to the increased river elevations.

The changes due to a water level elevation across the site of approximately 1006.85 ft compared to the pre-flood groundwater elevation of approximately 990 ft will be evaluated with respect to each structure.

Table 2-6 – Groundwater and River Level Elevations

Date	12/10/2010	3/22/2011	6/4/2011	9/1/2011
River Elevation[A]	**993.994**	**995.33**	**1002.86**	**1002.18**
Monitoring Well ID	**Groundwater Elevation (ft)**			
MW-1A	990.76	989.15	998.7	999.55
MW-1B	990.74	989.12	998.7	999.54
MW-2A	991.18	990.12	998.55	998.93
MW-2B	991.23	990.14	998.74	999.2
MW-3A	990.93	990.82	998.25	998.77
MW-3B	991.07	990.77	998.15	998.68
MW-4A	991.5	990.85	999.75	1000.4
MW-4B	991.48	990.73	999.63	1000.23
MW-5A	991.88	991.18	1000.15	1000.67
MW-5B	991.81	991.14	1000.12	1000.6
MW-6	991.71	992.08	1000.45	1001.13
MW-7	991.32	990.89	999.26	999.98
MW-9	990.82	989.28	998.68	999.49
MW-10	991.16	989.53	998.98	999.83
MW-11	991.21	989.93	998.88	999.48

[A] - River elevations include FCS data and interpolated stages between Omaha and Blair and between Omaha and Decatur, Nebraska.

- Is there observable ground subsidence?
- Is there observable pavement subsidence?
- Is there observable soil piping (sand boils, sinkholes)?

3.4 Identified Potential Failure Modes

The assessment teams identified 15 potential Triggering Mechanisms relative to the 2011 flood and FCS site inundation that could have materially and negatively impacted FCS structures. Once the Triggering Mechanisms were identified, PFMs that could develop as a result of those mechanisms were identified. A list of identified Triggering Mechanisms and associated PFMs is provided in Table 3-2.

Table 3-2 – Triggering Mechanisms and Potential Failure Modes

Triggering Mechanism No.	Triggering Mechanism	PFM No.	Potential Failure Mode
1	River Bank Erosion/Scour	1a	Undermining shallow foundation/slab
		1b	Loss of lateral support for pile foundation
		1c	Undermined buried utilities pipes/cables
		1d	Additional lateral force on piles
2	Surface Erosion	2a	Undermining shallow foundation/slab
		2b	Loss of lateral support for pile foundation
		2c	Undermined buried utilities
3	Subsurface Erosion/Piping	3a	Undermining and settlement of shallow foundation/slab (due to pumping)
		3b	Loss of lateral support for pile foundation (due to pumping)
		3c	Undermined buried utilities (due to pumping)
		3d	Undermining and settlement of shallow foundation/slab (due to river drawdown)
		3e	Loss of lateral support for pile foundation (due to river drawdown)
		3f	Undermined buried utilities (due to river drawdown)
		3g	Sinkhole development due to piping into karst voids
4	Hydrostatic Lateral Loading (water loading on structures)	4a	Overturning
		4b	Sliding
		4c	Wall failure in flexure
		4d	Wall failure in shear
		4e	Excess deflection
5	Hydrodynamic Loading	5a	Overturning
		5b	Sliding
		5c	Wall failure in flexure
		5d	Wall failure in shear
		5e	Damage by debris
		5f	Excess deflection
6	Buoyancy, Uplift	6a	Fail tension piles

Table 3-2 – Triggering Mechanisms and Potential Failure Modes			
Triggering Mechanism No.	**Triggering Mechanism**	**PFM No.**	**Potential Failure Mode**
	Forces on Structures	6b	Cracked slab, loss of structural support
		6c	Displaced structure/broken connections
7	Soil Collapse (first time wetting)	7a	Cracked slab, differential settlement of shallow foundation, loss of structural support
		7b	Displaced structure/broken connections
		7c	General site settlement
		7d	Piles buckling from down drag
8	Soil Solutioning	8a	Not applicable
9	Swelling of Expansive Soils	9a	Cracked slab, differential heave of shallow foundation, loss of structural support
		9b	Displaced structure/broken connections
		9c	Fail tension piles
		9d	Additional lateral force on below-grade walls
10	Machine/Vibration-Induced Liquefaction	10a	Cracked slab, differential settlement of shallow foundation, loss of structural support
		10b	Displaced structure/broken connections
		10c	Additional lateral force on below-grade walls
		10d	Pile/pile group instability
11	Loss of Soil Strength due to Static Liquefaction or Upward Seepage	11a	Cracked slab, differential settlement of shallow foundation, loss of structural support
		11b	Displaced structure/broken connections
		11c	Additional lateral force on below-grade walls
		11d	Pile/pile group instability
12	Rapid Drawdown	12a	River bank slope failure and undermining surrounding structures
		12b	Lateral spreading
13	Submergence	13a	Corrosion of underground utilities
		13b	Corrosion of structural elements
14	Frost Effects	14a	Heaving, crushing, or displacement
15	Karst Foundation Collapse	15a	Piles punching through karst voids due to additional loading

3.5 Initial Screening of Potential Failure Modes

A summary of Triggering Mechanisms and associated PFMs by structure is presented in Attachment 4. Structures to be assessed were selected and prioritized by OPPD and included buildings, process structures, equipment foundations, tank foundations, and electrical towers (structures). In Attachment 4, the structures are grouped into three categories:

- Class I structures
- Non-class I structures inside the PA
- Non-class I structures outside the PA

PFMs judged by the assessment teams to be credible based on initial screening are labeled "C" in Attachment 4. Failure modes deemed non-credible are labeled NC in Attachment 4, and failure modes that do not apply to a particular structure are labeled NA in Attachment 4.

Attachment 4 presents the results of initial screening. As more information becomes available, each PFM will be reevaluated and rerated as appropriate. The results of the PFM analysis for each structure and system are presented in Sections 5.0 and 6.0 of this Assessment Report.

3.6 Potential Failure Modes Deemed Non-Credible for All Structures

The results of the field observations combined with review of FCS design documents indicated that some of the potential Triggering Mechanisms and/or their associated PFMs listed in Table 3-2 did not occur or were not deemed credible as a result of the 2011 flood. For example, site investigations revealed no evidence of bank scour along the east boundary of the site. Therefore, all the PFMs associated with the Triggering Mechanism of river scour/bank erosion were determined to be non-credible because the Triggering Mechanism did not occur. The PFMs described in Table 3-3 were judged to be non-credible for all FCS structures evaluated with the exception of the PFMs associated with Triggering Mechanism 9, which was judged to be non-credible for only Priority 1 Structures. Table 3-3 shows Triggering Mechanisms 10, 12, 13 and 14 as non-credible, note that these Triggering Mechanisms were deemed non-credible after the completion of the assessments for Priority 1 Structures and before the assessment of Priority 2 Structures. The rationale for their elimination from the list of CPFMs is also presented.

Table 3-3 – Potential Failure Modes Determined to be Non-Credible

Identifier	Potential Failure Mode	Rationale for Elimination
Triggering Mechanism 1 – River Bank Erosion/Scour		
PFM 1a	Undermining shallow foundation/slab	Triggering Mechanism 1 did not occur: • Bathymetric survey of the river channel and banks indicated no observable sloughing, scouring, or other signs of bank erosion. • Visual observations of the river bank indicated no sloughing, scouring, or other signs of bank erosion. • Bank stabilization features installed by USACE are robust, and there is no known major bank failure as a result of 2011 flooding. • The river is back to nominal normal levels, and the Triggering Mechanism was not observed.
PFM 1b	Loss of lateral support for pile foundation	
PFM 1c	Undermined buried utilities pipes/cables	
PFM 1d	Additional lateral force on piles	
Triggering Mechanism 3 – Subsurface Erosion/Piping		
PFM 3d	Undermining and settlement of shallow foundation/slab (due to river drawdown)	The river is back to nominal normal levels, and the PFMs were not observed.
PFM 3e	Loss of lateral support for pile foundation (due to river drawdown)	
PFM 3f	Undermined buried utilities (due to river drawdown)	

Table 3-3 – Potential Failure Modes Determined to be Non-Credible

Identifier	Potential Failure Mode	Rationale for Elimination
PFM 3g	Sinkhole development (due to piping into karst voids)	Karst voids are filled with water. There is no head differential (gradient) to initiate this type of soil erosion.
Triggering Mechanism 8 – Soil Solutioning		
PFM 8a	Various	Triggering Mechanism 8 did not occur: • Mineralogy of local soils is not susceptible to solutioning.
Triggering Mechanism 9 – Swelling of Expansive Soils		
PFM 9a	Cracked slab, differential heave of shallow foundation, loss of structural support	Triggering Mechanism 9 did not occur for Priority 1 Structures: • Highly expansive soils are not present under the Priority 1 Structures. Structures are founded either on non-expansive select fill or on non-expansive native granular soils (pile-supported structures). **Note:** These PFMs were analyzed further for Priority 2 Structures where, in some cases, expansive soils are present.
PFM 9b	Displaced structure/broken connections	
PFM 9c	Fail tension piles	
PFM 9d	Additional lateral force on below-grade walls	
Triggering Mechanism 10 – Machine/Vibration Induced Liquefaction		
PFM 10a	Cracked slab, differential settlement of shallow foundation, loss of structural support	Triggering Mechanism 10 did not occur: • Groundwater is back to nominal normal levels, and the PFMs were not observed.
PFM 10b	Displaced structure/broken connections	
PFM 10c	Additional lateral force on below-grade walls	
PFM 10d	Pile/pile group instability	
Triggering Mechanism 12 – Rapid Drawdown		
PFM 12a	River bank slope failure and undermining surrounding structures	Triggering Mechanism 12 did not occur: • Groundwater is back to nominal normal levels, and the PFMs were not observed.
PFM 12b	Lateral spreading	
Triggering Mechanism 13 – Submergence		
PFM 13a	Corrosion of underground utilities	Triggering Mechanism 13 did not occur: • The structures were not subjected to a corrosive environment that would be considered beyond normal conditions.
PFM 13b	Corrosion of structural elements	
Triggering Mechanism 14 – Frost Effects		
PFM 14a	Heaving, crushing, or displacement	Triggering Mechanism 14 did not occur: • Prior to ground freezing, the groundwater returned to nominal normal levels.
Triggering Mechanism 15 – Karst Foundation Collapse		
PFM 15a	Piles punching through karst voids due to additional loading	Triggering Mechanism 15 did not occur: • Piles were driven or drilled to an elevation below the deepest karst/erosional feature. Explorations for the design/construction extended into bedrock. No voids exist below the pile tips. Additional vertical load due to soil down drag is minimal compared to the "baseline" vertical load.

3.7 Assessment Methods

Table 3-4 lists the various methods that might be used to determine the significance of the potential of failure for any of the structures. The methods included visual observations of the structures and civil works, field surveys, and geophysical and geotechnical investigations. Field teams composed of structural, civil, and geotechnical engineering professionals examined the structures as floodwater receded. These investigations were based on detailed checklists, as noted in Section 3.3. The results of the visual observations were supplemented with elevation surveys and geophysical and geotechnical investigations. Note also that Table 3-4 lists methods for Triggering Mechanisms 10, 12, 13, and 14; however, these Triggering Mechanisms were deemed non-credible after the completion of the assessments for Priority 1 Structures and before the assessment of Priority 2 Structures.

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
1. River Bank Erosion/Scour	a. Undermining shallow foundation/slab	These PFMs were determined to be non-credible.			
	b. Loss of lateral support for pile foundation				
	c. Undermined buried utilities pipes/cables				
	d. Additional lateral force on piles				
2. Surface Erosion	a. Undermining shallow foundation/slab	[*Note*: these actions were taken for each PFM.] Interview OPPD staff. Review plans and specifications to identify pertinent design and construction details needed to define pre-flood conditions. Review OPPD Condition Reports to determine changes and modifications since construction. Review flood data including observed flow conditions, depths, and velocities.	Observe surface condition for erosion, broken pavement, depressions, gullies, and other signs of distress, and hand probe area adjacent to structures.	Look for settlement of slab, cracks in foundation and walls, tilt, or settlement of foundation.	
	b. Loss of lateral support for pile foundation		Observe soil conditions around structure for settlement.	Observe pile-supported slab for cracking or excessive deflection.	

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
2. Surface Erosion (continued)	c. Undermined buried utilities		Observe surface condition for erosion, broken pavement, depressions, gullies, and other signs of distress, and hand probe area adjacent to structures. Conduct video inspection of open conduits and pipe in soil if accessible or as possible.		
3. Subsurface Erosion/Piping	a. Undermining and settlement of shallow foundation/slab (due to pumping)		Observe surface condition around buildings for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations or slabs.	Observe settlement of slabs, cracks in foundation, or settlement of foundation.	Test for voids using ground penetrating radar (GPR). Hydro-excavate suspect areas where feasible.
	b. Loss of lateral support for pile foundation (due to pumping)		Observe soil conditions around structure for settlement.		Sample areas adjacent to structures using standard penetration test (SPT) or cone penetration test (CPT) methods as appropriate.

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
3. Subsurface Erosion/Piping (continued)	c. Undermined buried utilities (due to pumping)		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Conduct video inspection of open conduits and pipe in soil if accessible or as possible. Inspect utility manholes (MHs) if possible. Identify MH penetrations that leak; look for sediment in MH bottom and in pumped water.	Observe soil conditions at utilities for settlement or lost soil material.	Test for voids using GPR. Hydro-excavate suspect areas where feasible. Open test pit where feasible.
	d. Undermining and settlement of shallow foundation/slab (due to river drawdown)		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe soil conditions around structure for settlement of slab, cracks in foundation, or settlement of foundation.	Test for voids using GPR. Hydro-excavate suspect areas where feasible.
	e. Loss of lateral support for pile foundation (due to river drawdown)		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe soil conditions around structure for settlement.	Sample areas adjacent to structures using SPT or CPT methods as appropriate.

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
3. Subsurface Erosion/Piping (continued)	f. Undermined buried utilities (due to river drawdown)		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Conduct video inspection of open conduits and pipe in soil if accessible or as possible. Inspect utility MHs if possible. Identify MH penetrations that leak; look for sediment in MH bottom and in pumped water.	Observe soil conditions at utilities for settlement or lost soil material.	Test for voids using GPR. Hydro-excavate suspect areas where feasible. Open test pit where feasible.
	g. Sinkhole development (due to piping into karst voids)	This PFM was determined to be non-credible.			
4. Hydrostatic Lateral Loading (water loading on structures)	a. Overturning		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of movement.	
	b. Sliding		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of movement.	
	c. Wall failure in flexure		Survey/monitor elevation of designated points on foundations.	Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	
	d. Wall failure in shear		Survey/monitor elevation of designated points on foundations.	Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
4. Hydrostatic Lateral Loading (water loading on structures) (continued)	e. Excess deflection		Survey/monitor elevation of designated points on foundations.	Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	
5. Hydrodynamic Loading	a. Overturning		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of high water exposure or structure movement.	
	b. Sliding		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of high water exposure or structure movement.	
	c. Failure in flexure			Observe exposed structure for signs of high water. Observe exposed structural elements for signs of cracking, water leakage, or excessive (visible) deflection.	
	d. Failure in shear			Observe exposed structure for signs of high water. Observe exposed structural elements for signs of cracking, water leakage, or excessive (visible) deflection.	
	e. Damage by debris			Observe exposed structure for signs of high water or impact abrasions/damage from debris.	

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
5. Hydrodynamic Loading (continued)	f. Excess deflection			Observe exposed structure for signs of high water. Observe exposed structural elements for signs of cracking, water leakage, or excessive (visible) deflection.	
6. Buoyancy, Uplift Forces on Structures	a. Failed tension piles			Observe pile-supported slabs for cracking, upward deflection.	
	b. Cracked slab, loss of structural support		Observe perimeter grade condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe pile supported slabs for cracking or upward deflection.	Hydro-excavate suspect areas.
	c. Displaced structure/broken connections		Observe perimeter grade condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe structures for cracking, broken members, or other signs of structural distress.	
7. Soil Collapse (first time wetting)	a. Cracked slab, differential settlement of shallow foundation, loss of structural support		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe soil conditions around structure for settlement of slab, cracks in foundation, or settlement of foundation.	Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
7. Soil Collapse (first time wetting) (continued)	b. Displaced structure/broken connections		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe structures for cracking, broken members, or other signs of structural distress.	Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.
	c. General site settlement		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.		Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.
	d. Piles buckling from down drag			Observe pile-supported slabs for cracking or downward deflection.	Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.
8. Soil Solutioning	a. Not applicable	This PFM was determined to be non-credible.			

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
9. Swelling of Expansive Soils	a. Cracked slab, differential heave of shallow foundation, loss of structural support		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe foundations for cracking and/or deflection from swelling.	Sample areas adjacent to structures using Shelby Tube sampling and laboratory analysis as appropriate
	b. Displaced structure/broken connections		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe structures for cracking, broken members, or other signs of structural distress.	Sample areas adjacent to structures using Shelby Tube sampling and laboratory analysis as appropriate.
	c. Fail tension piles			Observe pile-supported slabs for distress.	
	d. Additional lateral force on below-grade walls			Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	Sample areas adjacent to structures using Shelby Tube sampling and laboratory analysis as appropriate.
10. Machine/ Vibration-Induced Liquefaction	a. Cracked slab, differential settlement of shallow foundation, loss of structural support		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe foundations for cracking and/or deflection from swelling.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
10. Machine/ Vibration-Induced Liquefaction (continued)	b. Displaced structure/broken connections		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe structures for cracking, broken members, or other signs of structural distress.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	c. Additional lateral force on below-grade walls			Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	d. Pile/pile group instability			Observe pile-supported slabs for cracking, downward deflection. Test for voids using GPR.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
11. Loss of Soil Strength due to Static Liquefaction or Upward Seepage	a. Cracked slab/differential settlement of shallow foundation/loss of structural support		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe foundations for cracking and/or deflection from swelling. Survey/monitor elevation of designated points on foundations.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
11. Loss of Soil Strength due to Static Liquefaction or Upward Seepage (continued)	b. Displaced structure/broken connections		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe structures for cracking, broken members, or other signs of structural distress.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	c. Additional lateral force on below-grade walls			Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	d. Pile/pile group instability			Observe pile-supported slabs for cracking, downward deflection.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.

Table 3-4 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
12. Rapid Drawdown	a. River bank slope failure and undermining surrounding structures		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe soil conditions around structure for eroded or lost material, settlement of slab, cracks in foundation, or settlement of foundation.	Install and monitor inclinometers. Hydro-excavate suspect areas.
	b. Lateral spreading		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe site soils conditions for signs of soil movements or spreading.	Install and monitor inclinometers.
13. Submergence	a. Corrosion of underground utilities	Review cathodic protection records.	Conduct video inspection of open conduits and pipe if accessible or as possible.		
	b. Corrosion of structural elements			Observe exposed structural elements for signs of rust, degraded material, or other signs of corrosion.	
14. Frost Effects	a. Heaving, crushing, or displacement				Test soil properties.
15. Karst Foundation Collapse	a. Piles punching through karst voids due to additional loading	This PFM was determined to be non-credible.			