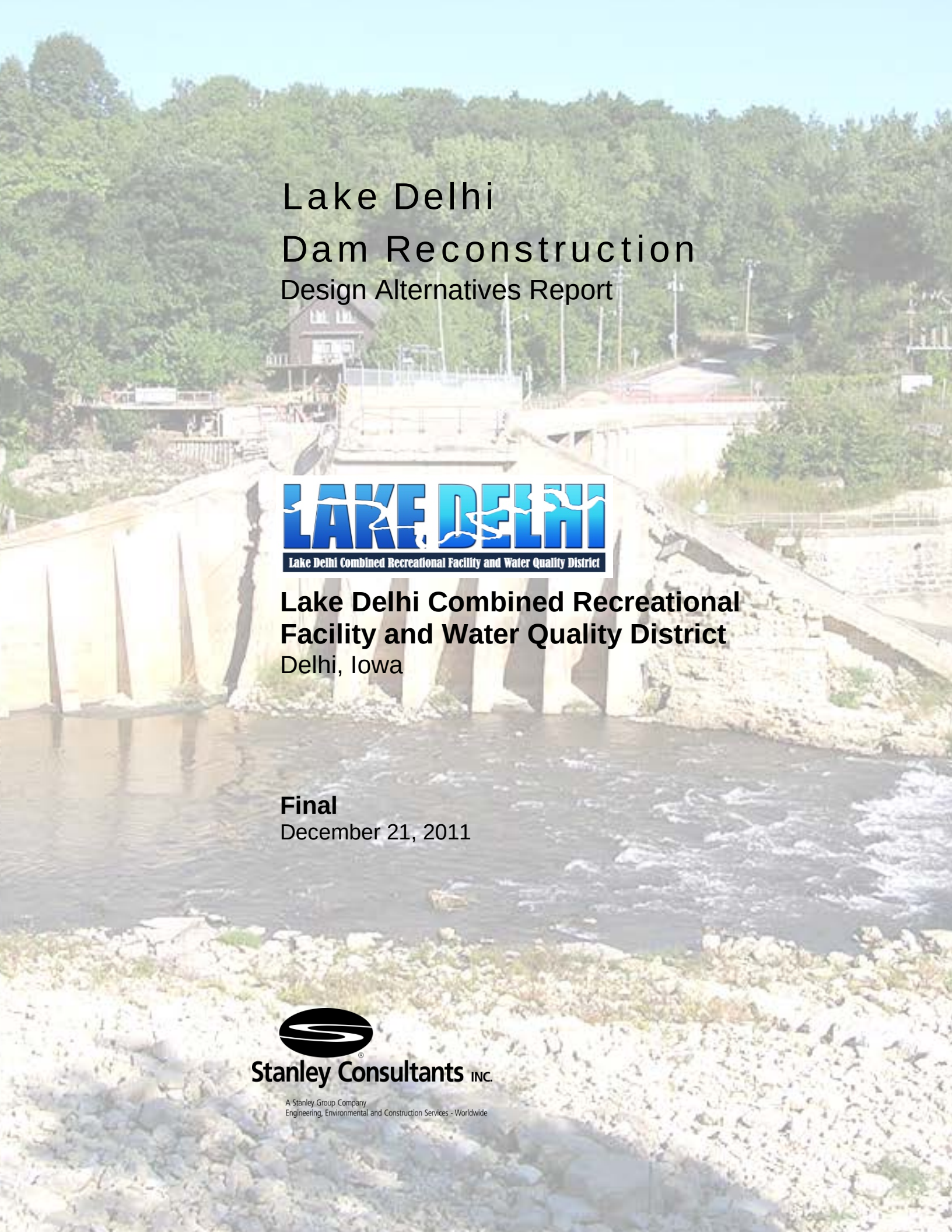

Lake Delhi
Dam Reconstruction
Design Alternatives Report

Lake Delhi Combined Recreational Facility and Water Quality District
Delhi, Iowa

Final
December 21, 2011

Stanley Consultants INC.
A Stanley Group Company
Engineering, Environmental and Construction Services - Worldwide

Lake Delhi Dam Reconstruction

Design Alternatives Report

Lake Delhi Combined Recreational Facility and Water Quality District

Delhi, Iowa

Final

December 21, 2011

LICENSED PROFESSIONAL ENGINEER
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IOWA

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Iowa.

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License number 20164

My license renewal date is December 31, 2013.

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Executive Summary

Lake Delhi Dam is located on the Maquoketa River in Delaware County, Iowa. The dam is maintained and managed by the Lake Delhi Combined Recreational Facility and Water Quality District (District). During the flood event of July 23–24, 2010, the dam's southern earthen embankment was overtopped and fully eroded and the concrete spillway gates were damaged. Floodwaters also infiltrated and seeped through a section of the northern embankment.

Stanley Consultants was selected by the District to perform analysis and preliminary design for the dam's reconstruction and restoration of the Lake Delhi Dam pool.

The objectives of this phase of the project are:

- Provide documentation of the existing condition of Lake Delhi Dam.
- Collect sufficient data to perform technical analysis and preliminary design of the dam reconstruction.
- Review regulatory requirements for dam reconstruction and present findings from the archaeological survey of the lake area.
- Develop and review alternatives for reconstructing Lake Delhi Dam and bringing the dam into compliance with current dam safety standards.
- Provide recommendations for final design and construction.
- Provide a preliminary estimate of construction costs and schedule.

This report provides a summary of findings from the surveys, research, inspection, technical analysis, and preliminary design performed to satisfy the project objectives.

Lake Delhi Dam consists of a series of distinct structures and features; all of which were evaluated in consideration of reconstruction of the dam, restoration of the lake, and future maintenance and operation of the dam.

Field investigation and data collection programs were completed to obtain the information and data required for assessment of the condition of existing structures and equipment and developing and evaluating conceptual designs.

Engineering analyses were completed to establish design requirements for reconstruction. These analyses established engineering parameters that will be utilized in design of repair and construction features, as well as minimum loading conditions for meeting current dam safety and design standards.

A detailed hazard classification analysis was completed and results indicate that a reconstructed Lake Delhi Dam most closely matches the DNR's Moderate Hazard Classification.

The Stanley Consultants design team met with District representatives for an Alternatives Development "Brainstorming" Session and objectives for the reconstruction project were established.

Project objectives were used as the framework for development of potential reconstruction alternatives for Lake Delhi Dam. Conceptual designs were used to estimate construction costs and evaluate each alternative relative to project objectives.

Based on the project objective evaluation and cost comparison, reconstruction alternatives were selected for incorporation into the "recommended project." A preliminary construction cost estimate and construction schedule were then developed for the recommended project.

It is recommended that construction be split into two phases. The first phase would involve restoration and upgrading of the existing powerhouse and gated spillway structure (north side). The second phase would involve reconstruction of the eroded southern embankment and construction of a new spillway to increase discharge capacity.

From the preliminary scheduling it was determined that construction could be accomplished in one construction season but assumes normal weather conditions and an experienced contractor with sufficient resources.

The construction cost estimate for the "recommended project" at this conceptual stage of design is approximately $11.9 million.

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Section 1

Project Description

1.1 General

Lake Delhi Dam is located on the Maquoketa River in Delaware County, Iowa. The dam is maintained and managed by the Lake Delhi Combined Recreational Facility and Water Quality District (District). During the flood event of July 23–24, 2010, the dam's southern earthen embankment was overtopped and fully eroded and the concrete spillway gates were damaged. Floodwaters also infiltrated and seeped through a section of the northern embankment.

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This report provides a summary of findings from the surveys, research, inspection, technical analysis, and preliminary design performed to satisfy the project objectives. Lake Delhi Dam consists of a series of distinct structures and features; all of which were evaluated in consideration

of reconstruction of the dam, restoration of the lake, and future maintenance and operation of the dam.

1.2 Description of Features

Exhibit 1 in Appendix F displays the layout of the dam features. A description of the dam's major structures and features evaluated during the alternatives analysis is provided by the following:

Lake Area

Prior to the 2010 dam breach, Lake Delhi extended approximately 7 miles upstream of the dam on the Maquoketa River, with a surface area of approximately 440 acres and a storage volume of 3790 acre-feet. During normal flow conditions, the pool elevation was maintained at elevation 896.3 ft-msl.

North Embankment

The north embankment is located between the north river bank of the Maquoketa River and the Powerhouse Structure and consists of earthen embankment with retaining walls supporting a concrete roadway (230th Avenue). The upstream retaining wall is a precast concrete "bin-type" retaining wall and the downstream retaining wall is a curved reinforced concrete wall. Both walls connect to the Powerhouse Structure.

Powerhouse Structure

The powerhouse structure was built in the 1920s by Interstate Power. Power generation was ceased in 1973, but the wicket gates continued to be used for passing normal flows at the dam. The Powerhouse is a multi-level reinforced concrete structure consisting of three main rooms on three levels. The upper level is the control room, the middle level is the turbine room, and the lower level is the mechanical room. The roof of the powerhouse is a concrete bridge deck with an operator platform separated from the bridge deck by fencing and railing on the upstream side. The top of the bridge deck is at elevation 904.7 ft-msl. Two turbine intakes with a trash rake system are located on the upstream side of the structure. Flow through these intakes is controlled by the wicket gates which were used to discharge normal flows at the dam.

Gated Spillway Structure

The gated spillway structure is located adjacent to the Powerhouse Structure and includes three concrete ogeee spillways separated by concrete spillway piers and abutment walls, with a concrete bridge deck over the top. Three vertical steel slide gates and hoisting equipment are located on a platform on the upstream side of the structure, separated from the bridge deck by fencing and railing. The crest of the ogee spillway is at elevation 879.8 ft-msl, approximately 16.5 feet below the normal pool elevation. The slide gates were usually kept closed and only opened to pass debris and flood magnitude flows at the dam. A large quantity of riprap was deposited upstream of the spillway gates in 2009.

Stilling Basin

The stilling basin is located on the downstream side of the gated spillway structure at the river channel bottom. The concrete stilling basin floor is at an elevation of roughly 849 ft-msl, but is currently buried under approximately 10 feet of silt. The stilling basin is bordered on the north and south by the North Downstream Abutment Wall and the South Buttress Wall, respectively.

South Buttress Wall

The south buttress wall is located on the south side of the gated spillway structure. Prior to the breach it tied into the southern earthen embankment. The wall is on the north side of the breach area and currently approximately 40 feet of the wall is exposed from the channel bottom up to the top of the concrete bridge deck. An abandoned narrow concrete fish ladder is fastened to the top of wall on the downstream side.

North Downstream Abutment Wall

The North Downstream Abutment Wall extends downstream from the Powerhouse Structure. The base and lower portion of the wall is reinforced concrete and the upper portion is masonry block. There is a gravel and grassed access area behind the wall that is even with the turbine room floor of the powerhouse.

South Embankment Area

The South Embankment Area was breached during the 2010 flood. The earthen embankment was almost fully washed away, exposing the river channel bottom. Following the breach, the Iowa Department of Natural Resources (DNR) completed a channel stabilization project that involved installing riprap along the channel bottom and south river bank. There is a concrete cutoff wall that is currently exposed on the south river bank. This cutoff wall used to extend to the South Buttress Wall, but this portion was washed away in the 2010 flood.

Information pertaining to the current condition and reconstruction alternatives for the dam features and structures is provided in subsequent sections of the report.

Section 2

Field Investigations and Data Collection

2.1 General

Field investigation and data collection programs were completed to obtain information for evaluating existing structures and equipment and developing conceptual design. Collected data were utilized in engineering analyses and evaluations to determine upgrades necessary to meet current requirements of regulating agencies and modern dam design standards. These data were also used in the development and evaluation of conceptual designs for dam repair/reconstruction alternatives. The investigation and data collection programs included:

- Topographic Survey.
- Property Research.
- Geotechnical Investigations.
- Structural Investigations.
- Electrical Investigations.
- Mechanical Investigations.
- Archeological Reconnaissance.
- Permitting Requirement Review.

2.2 Topographic Survey

Topographic survey of the dam area was performed by Gibbs Engineering and Surveying from Manchester, Iowa (Gibbs). The survey was performed over several weeks in June and early July 2011. Gibbs issued AutoCAD drawings of the topographic survey which were used in the preliminary design of reconstruction alternatives and development of reconstruction alternative exhibits.

The coordinate system and vertical datum for the topographic survey are:

- Coordinate System: North American Datum of 1983 (NAD 83), State Plane, Iowa North Zone (1401).
- Vertical Datum: North American Vertical Datum of 1988 (NAVD 88).

In addition to NAVD 88, some reference documents and drawings use the National Geodetic Vertical Datum of 1929 (NGVD 29) as well as a local datum which converts to NGVD 29 by adding 774.8 ft to the local datum elevation. NAVD 88 is approximately 0.1 feet lower than NGVD 29 in the Lake Delhi region. Due to the number of references used in this report, the lack of datum definition on several documents, the small difference in vertical datums, and the preliminary stage of design, conversions were not made to a single datum. A single datum (likely NAVD 88) would be used in final design and during construction.

Light Detection and Ranging (LiDAR) data was used for topography outside of the Gibbs survey area (mostly used in hydrologic and hydraulic analysis). LiDAR uses aircraft mounted light-emitting laser scanners to obtain high accuracy elevation data. The Iowa Department of Natural Resources (DNR) has obtained LiDAR data for the entire state of Iowa. LiDAR in the Lake Delhi region was flown post-breach so contains the topography of Lake Delhi below the normal pool which was helpful in developing hydraulic model cross-sections. The DNR LiDAR uses NAVD 88 so is on the same vertical datum as the Gibbs survey.

2.3 Property Research

Gibbs also developed an exhibit of property boundaries in the dam area. The exhibit is not a legal property survey but provides a good depiction of property line locations relative to the dam and surrounding area. Development of the property exhibit included:

- Reviewing Gibb's previous surveys and extracting plats, deeds, and easements.
- Reviewing Gibb's previous computer drawings and copying previous line/survey work
- Generating a property line exhibit compiling previous Gibb's work.
- Conducting research at County courthouse and obtaining nearby plats, surveys, or deeds. Obtaining court cases from the clerk of court referencing the recent Rocky Nook, Lake Delhi Recreation Association property dispute.
- Drawing in the geometry of missing plats into property line exhibit.
- Obtaining coordinates of accessible property pins during topographic survey.
- Comparing surveyed property pins to property line exhibit and adjusting lines as needed to reflect surveyed property pins.

Property and easement requirements for the project will be better defined as design develops. A legal survey will be completed at this time.

2.4 Geotechnical Investigation

A geotechnical investigation was completed by Braun Intertec. A boring plan was developed by Stanley Consultants, advancing 12 borings along the proposed alignment of the dam features. Borings ST-1 and ST-2 were drilled through the north embankment. Borings ST-4 through ST-7 were drilled within the breach limits of the south embankment. Borings ST-8 through ST-10 were drilled through the embankment that remains south of the breach. Borings ST-11 and ST-12 were drilled through the existing powerhouse and spillway bridge deck. All borings were advanced to sufficient depths to allow analysis and evaluation of soil and bedrock foundations for embankment/structural stability and seepage.

Soil samples were collected with split spoon and Shelby tube samplers. Blow counts (N-Values) were recorded by the Braun Intertec drilling crew. Soil samples were classified and tested. Testing on the soil boring samples included moisture content and dry density, Atterberg Limits, unconfined compression testing, and gradations.

Boring ST-11 and ST-12 were advanced through the existing walls and piers of the powerhouse and spillway. Continuous core samples were collected in the concrete and underlying bedrock foundation. Percent recovery and Rock Quality Designation (RQD) were recorded by the Braun Intertec drilling crew for all bedrock core collected. The bedrock core was classified and representative samples tested for unconfined strength.

Borings were also drilled at several properties in the vicinity of the dam to determine the extents and accessibility of loess and till materials for potential use in reconstruction of the earthen portion of the dam. Borings were advanced on the Wilson, Freiburger, and Harbach properties. Soil samples collected from the borings were classified and tested for moisture content, compaction testing, and Atterberg Limits.

Braun Intertec's preliminary geotechnical report describing the geotechnical investigations and presenting results, including boring logs and laboratory test results is included in Appendix C.

2.5 Structural Investigation

A structural inspection/evaluation of Lake Delhi Dam was performed by Stanley Consultants on September 23, 2010 after the July 2010 flood. A copy of the complete report is included in Appendix A.

Additional investigation of concrete structures was completed by Stanley Consultants engineers during their site visit on November 9, 2011. During the investigation concrete deterioration and spalling were observed in many areas of the powerhouse and spillway structures. Concrete on the surface of the spillway and lower portion of the bridge piers was found severely deteriorated. Concrete surrounding the lift gate slots was severely spalled.

As part of the current structural investigation, Stanley Consultants performed a review of the spillway and powerhouse stability analysis completed by Ashton-Barnes Engineering in 1997. In the 1997 Ashton-Barnes report, it was concluded that both the ogee spillway and powerhouse turbine bay section were stable for the normal flow condition (with or without an ice loading).

The dam was not found to have adequate stability for the Probable Maximum Flood (PMF) condition.

In review of the Ashton-Barnes analysis, it was found that:

- Ashton-Barnes categorized Lake Delhi Dam as a low hazard potential and the structures were analyzed accordingly.
- The analysis did not consider rock or silt deposits on the upstream face of the spillway gate structure which applies a lateral driving load on the structure. The Ashton-Barnes report noted but did not analyze the load from significant silt build-up at the upstream face. Currently, a significant amount of riprap stone protection is at the upstream face from a 2009 project.
- The 1997 Ashton-Barnes analysis was based on Federal Energy Regulatory Commission (FERC) criteria.
- Water elevations were based on old hydrology data and hydraulic analysis.
- Bedrock - dam concrete bonding strength of 40 pounds per square inch (psi) and a friction angle of 35 degrees were used in the analysis. These were assumed parameters with no bedrock coring verification.

Based upon the field investigations and the results of the Ashton Barnes analyses review, additional investigation and analyses were completed. These included taking the two concrete core borings through the powerhouse and spillway structures to observe the bedrock/concrete interface and to evaluate the condition and strength of the foundation bedrock. In addition, stability analyses were completed of the powerhouse and spillway structures, utilizing recent hydrologic data, dam hazard classification and following the requirements of the FERC and U.S. Army Corps of Engineers (USACE) dam design guidelines.

A concrete coring and testing program was developed to evaluate the subsurface condition of the concrete. Stanley located sites for obtaining concrete cores to evaluate the composition and condition of the concrete below the surface. Braun Intertec completed the concrete coring. Representative cores were selected for unconfined compressive testing and petrographic analyses. Locations of core holes, as well as photographs of the cores and core holes, and laboratory test results are included in Appendix C.

The lower downstream face of the spillway structure and the downstream stilling are not observable due to riprap and silt deposits, respectively. These were found to be in relatively good condition during the 2008 J.F. Brennan Co. underwater inspection; but because the July 2010 flood subjected the dam to conditions that could have undermined the downstream edge of the concrete structures, additional inspection and evaluation will be required during construction when the area is dewatered and riprap and silt removed.

Findings from the 2011 site visit and investigation that were not discussed in the 2010 Stanley Consultants Inspection Report are provided by the following:

- Downstream north abutment wall, upper portion – stone block wall has severely deteriorated.

Downstream North Abutment Wall
Figure 2-1

- Significant seepage through powerhouse concrete slab roof was observed.

Powerhouse Roof Seepage
Figure 2-2

- A hole was observed at the front of the downstream north embankment wall. This indicated that the embankment material behind the retaining wall or foundation for the wall may have been eroded by overtopping flows or piping flows.

Material Loss Downstream of North Embankment Wall
Figure 2-3

2.6 Electrical Investigation

Power Distribution

The Lake Delhi Dam is currently served at 480/277V by three pole-mounted transformers located approximately 150 feet north of the powerhouse structure along 230th Avenue. The transformers and existing service drop to the powerhouse are owned by Alliant Energy. The existing utility meter and service disconnect switch, shown in Figure 2-4 below, are located on the north exterior wall of the powerhouse. The meter and disconnect switch are rated for outdoor environments and appear to be in good condition. The overhead service drop conductors also appear to be in good condition.

Meter and Main Service Disconnect
Figure 2-4

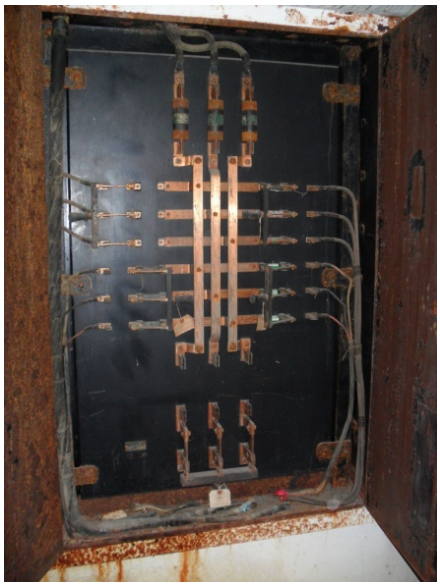
This exterior-mounted disconnect switch feeds an enclosed circuit breaker located in the stairwell of the powerhouse. The enclosed circuit breaker provides service to the main 480-volt distribution panelboard (Panel HP), located in the northwest room in the main floor of the powerhouse. A 30-kVA dry-type transformer steps the voltage down to 208/120-volt distribution for lighting and receptacle circuits in the powerhouse. A 208/120-volt distribution panelboard (Panel LP) is located next to panel HP and the step-down transformer. Figure 2-5 shows panel HP (right), the step-down transformer and panel LP (left). Also shown is an automatic transfer switch (center) not currently connected to the system.

Distribution Panelboards
Figure 2-5

The automatic transfer switch, panelboards, step-down transformer, enclosed circuit breaker, meter and service disconnect switch were installed within the last couple of years to update the electrical service to the dam to 480/277-volts. Prior to the installation of this equipment, the dam was served at 208/120-volts. The panelboards were both installed in NEMA 3R weatherproof enclosures and do not appear to have any water damage due to the flooding.

The 208/120-volt system is still present in the dam, although much of it has been disconnected. The 208/120-volt system was disabled following the 2010 flood event. The service disconnect switch and overcurrent protection, located on a utility pole just north of the powerhouse, is currently in the 'off' position and the service drop cables have been cut.

All of the distribution equipment from the disconnected 208/120-volt system is still present in the powerhouse, including two outdated fuse boxes and a distribution panelboard with no cover plate. One of the outdated fuse boxes is shown in Figure 2-6.



Outdated Fuse Box
Figure 2-6

The lighting and receptacle circuits in the powerhouse are being fed from the new panel LP at 120-volts. Prior to installation of the 480-volt service, these circuits were fed from the distribution panelboard with no cover plate. This panelboard is now being used as a junction box in order to utilize existing wiring in the facility, but to provide power from the new panel LP. The existing distribution panelboard with no cover plate is shown in Figure 2-7.

Distribution Panelboard with No Cover Plate
Figure 2-7

Trash Rake and Hydroelectric Equipment

The trash rake equipment, located on the top of the powerhouse structure, does not appear to have suffered significant water damage during the flooding. The PLC control cabinet does not have any indication of water damage; however, the conveyor equipment was underwater and may be damaged from the flooding. Figure 2-8 shows the existing trash rake system and trash rake PLC control cabinet.

Trash Rake Equipment
Figure 2-8

There does not appear to be any electrical equipment for pool level monitoring. There is an existing control panel, shown in Figure 2-9, labeled as 'Water Level Control Panel', which appears to have been used in conjunction with the hydroelectric generation equipment. There were not any field instruments observed in operation with this control panel.

Water Level Control Panel
Figure 2-9

The existing hydroelectric generation equipment is not operational. The control panels for each hydroelectric unit were inundated with water during the 2010 flood event and significant damage was observed to the battery systems and electronic control devices. Figure 2-10 shows one of the hydroelectric unit control panels. The main PLC cabinet for the hydroelectric system is located on the wall in the generator room and was also damaged by water during the flood event.

Hydroelectric Generator Control Panel
Figure 2-10

Each hydroelectric generator control panel also provides power to the associated wicket gate actuators. Currently, the wicket gates are operated manually, as the control system for the hydroelectric equipment and wicket gates are non-operational.

Lift Gates

The existing lift gates are operated via pushbutton stations located at each lift gate operator. During the 2010 flood event, gate 3 failed to open completely. The existing control equipment has no automatic operation capability and is partially exposed to the elements.

Emergency Generator System

A new automatic transfer switch was installed in the powerhouse when the 480/277-volt service was installed. There was a propane generator located on the site, but it is no longer present.

2.7 Mechanical Investigation

Lift Gates

There are three existing vertical Broome Gates that are original from 1927. During the 2010 flood, one of them did not open completely because the guide in the dam had deteriorated. The cable and drum hoist mechanisms for raising the gates had been recognized as needing replacement. Prior to the flood, the Owner purchased and received new electric motor driven screw actuators for the three existing gates, shown in Figure 2-11. The three existing gates, and gate guides at the dam need to be replaced for the gate system to function adequately.

New Gate Hoisting Mechanism
Figure 2-11

The wicket gates, from the two abandoned hydropower generators, had been maintained to provide minimum flow control. During the flood, the powerhouse was completely inundated and the electronic controls (PLC, battery backup, dc-motor, switches, etc.) were ruined. The wicket gates will require replacement of the existing actuator to bring the system into service. The actual gates and actuator arms will likely need refurbishment as well.

The HydroRake was installed in 2009 to remove the trash that accumulated on the two bar screens protecting the wicket gates. The raking system consists of two bar screens with a hydraulically operated trash raking arm mounted on a traveling chassis that discharges onto belt conveyors that remove the trash to the discharge of the spillway gates. The flood inundated this equipment except for the PLC control cabinet. To make this system operational again, a complete inspection of the equipment is required. At a minimum, the motors would need to be replaced, the local switches and sensors would need to be replaced, the submerged wiring would need to be replaced, and the hydraulic oil would need to be flushed. The belts, bearings, rollers, and hydraulic motor would need to be inspected, oiled, greased, or replaced as necessary. The trash rake is shown on Figure 2-12.

Trash Rake
Figure 2-12

2.8 Document Research

Several sets of historical photos, inspection reports, analyses, and construction drawings were obtained during the course of the reconstruction analysis. Unfortunately a complete set of dam construction drawings was never located. However plan, elevation, and section drawings of the powerhouse, gated spillway, and embankment were located and used to supplement the information obtained by the topographic survey of the dam area.

2.9 Prior Investigations

In addition to the investigations conducted during the alternatives analysis, the following prior investigation documents were reviewed:

- 1997 Ashton-Barnes inspection, stability, and spillway adequacy report.
- 2002 J.F. Brennan Co. underwater inspection report and videos.
- 2004 DNR inspection report.
- 2008 J.F. Brennan Co. underwater inspection report and videos
- 2009 DNR inspection report.
- 2010 Stanley Consultants inspection report.

With no pool, the majority of the dam structure was accessible for inspection during the alternatives analysis phase. However the large riprap installed on the upstream side of the dam and roughly 10 feet of silt deposited in the downstream stilling basin prevented inspection of the upstream spillway structure face and downstream stilling basin. These structures were inspected during the 2008 J.F. Brennan Co. underwater inspection. The underwater inspection found no major issues with the structure and recommended some minor repair of gate piers, the downstream stilling basin wall, and north wing wall of the stilling basin. No evidence of undermining was found at the upstream face of the gated spillway structure. Most of the area upstream of the spillway gates was 12 feet below the spillway crest and covered with riprap size

stone. However an area 20 feet below the spillway crest was found near the northernmost gate without revetment stone. This could have been the area near the dam's sluice pipes although the underwater inspection did not locate any pipe intakes on the upstream face. The underwater inspection report recommended filling the deepest portions of the upstream area with riprap due to "scouring."

The 2008 J.F. Brennan Co. and 2011 Stanley Consultants inspection reports have been included in Appendix A.

2.10 Archaeological Reconnaissance Survey

In September 2011, The Louis Berger Group, Inc. (LBG) completed an archaeological reconnaissance survey of the Lake Delhi area. The archaeological studies included a records review to identify potential resources within the former impoundment area followed by a field reconnaissance survey to investigate areas considered to have high potential for unreported archaeological sites. The study area included the Lake Delhi dam and all exposed land areas within the former impoundment area located at or below the former lake elevation level of 897 feet above mean sea level. The study area encompasses an estimated 448 acres.

No archaeological sites had been reported within the project area prior to the July 2010 dam failure. Four sites were recorded within the previous impoundment area during the fall of 2010 by Wapsi Valley Archaeology, Inc. (WVA) during archaeological monitoring for the installation of emergency erosion control structures at the Delhi dam location and upstream at Hartwick bridge.

The four sites included two historic building foundations and two historic artifact scatters associated with the 19th century town site of Hartwick. Prehistoric artifacts with evidence for Early to Middle Archaic, Late Archaic, Middle Woodland, and late prehistoric components were also collected from the four sites.

The LBG study includes a comprehensive records review, a condition assessment of the study area's Quaternary and Holocene valley landforms, and results of a reconnaissance level survey of those landforms.

LBG identified seven additional sites within the study area and redefined one of the sites first identified by WVA to segment one of the historic building foundations at Hartwick as a separate site. As a result, there are a total of 12 archaeological site reported for the study area. These include ten sites with evidence for prehistoric Native American occupations ranging from 8000 to 300 years before present (BP). Most of these sites (7 of 10) appear to be open habitation areas or settlements while one is a smaller habitation site situated within a natural rock shelter. Other prehistoric sites include an apparent fish weir structure and a lithic resource procurement site. Mid-19th century building foundations are represented at two separate locations near the former town site of Hartwick and are believed to be associated with the historic settlement that once existed at that location. One of these is believed to be the Hartwick saw mill which was the first building erected in Hartwick (by John Clark in 1849). Fragments of contemporary historic artifacts were identified at two sites that also produced prehistoric artifacts.

No burial sites were identified within the study area, but potential for unreported human burials is considered possible at the eight prehistoric habitation sites. None of the 12 sites has been evaluated for National Register eligibility. Additional reconnaissance survey is recommended for selected portions of the study area based on the results presented in this report. Additional site investigations are also recommended at all 12 sites as necessary for the purpose of gathering information about the nature, extent, and condition of the archaeological deposits present pursuant to an evaluation of National Register eligibility.

The full Archaeological Reconnaissance Survey Report is included in Appendix E.

2.11 Permitting Requirements

Permitting for the dam reconstruction will be through the USACE/ DNR Joint Permit process. During the detailed design phase an application package will be prepared for submittal to USACE with copies sent simultaneously to both the Floodplain and Sovereign Lands Section at DNR. Included in the submittal will be a separate packet with the forms and information specific to a Dam Construction Permit.

The archaeological survey report will be submitted with the application.

USACE Permit

Conversations with USACE have suggested that the project should qualify for the more streamlined Section 404 Nationwide Permit (NWP). During the design phase, an application will be submitted to the USACE demonstrating that the project meets the conditions for a NWP. USACE review times are typically less than a month. The Section 404 action will trigger the need to obtain Section 401 Water Quality Certification from DNR.

DNR – Section 401

Section 401 Water Quality Certification (aka 401 Cert) specifically addresses the project's potential impacts to water quality that will have to be avoided, minimized, and possibly mitigated.

DNR – Sovereign Lands

A Sovereign Lands Construction Permit will not be required for the project; however, as indicated above the Joint Application process will include a copy of the application to the Sovereign Lands Section for their review. This process will include a review within DNR by threatened & endangered (T&E) species staff and DNR fisheries personnel.

The T&E review will identify any state-listed plant or animal species known from the project area. It will be necessary to assess the likelihood that any of these species will be impacted by the project.

DNR – Floodplain Permit

Construction in a floodplain or floodway always requires a floodplain permit or an evaluation of floodplain issues, but with dam projects it is necessary to complete application forms and provide information specific to dam construction. For this project it will be necessary to

obtain a Construction Permit (Floodplain Development Permit). Submittal requirements include:

- Completed and signed Water Storage Permit Application.
- Two sets of certified plans.
- Engineering Design and Hydraulics and Hydrology Report.
- Soil & Foundation investigation report.
- Sedimentation rate assessment.
- Gated low-level outlet design.
- Hazard assessment.
- Summary of Engineering Data.

Cultural Resources

Along with the archaeological survey report, it will be necessary to develop a Programmatic Agreement with the State Historic Preservation Office (SHPO) that will provide a plan for avoiding, minimizing, or mitigating impacts to any significant resources encountered.

U.S. Fish & Wildlife Service (FWS)

FWS will be sent a Public Notice by USACE. FWS will review the project for the potential for the project to impact any federally-listed T&E species. Bald eagles are no longer a listed species but if any potential impacts are identified, application will be made to FWS for a Bald Eagle Permit.

The project area will be reviewed for the potential for federally-listed T&E species to occur in the area. If any potential exists, the Moline, Illinois Field Office of FWS will be contacted during preparation of the application. Any T&E concerns identified by FWS will be addressed in the application and the Moline office will be sent a copy of the application package at the same time it is submitted to the USACE.

Section 3

Engineering Analysis and Preliminary Design

3.1 General

Engineering analyses were completed to establish the requirements for the detailed design of the proposed dam repair and construction concepts. These analyses established engineering parameters that will be utilized in design of repair and construction features, as well as minimum loading conditions for meeting current dam safety and design standards. The engineering design parameters and loading conditions established by engineering analyses were utilized in completion of preliminary design of repair/reconstruction alternatives. Preliminary design established approximate sizing and construction of dam features used in comparative evaluation, preparation of preliminary cost estimates and determination of property/easement acquisition requirements.

Engineering analyses and preliminary design were completed for the following disciplines:

- Geotechnical.
- Structural.
- Hydrology/Hydraulics.

3.2 Geotechnical

Subsurface Investigation

A boring program was established to collect subsurface data necessary to evaluate construction of existing embankments, type and condition of dam foundation materials, and complete analysis of several preliminary design features. Borings were also advanced at several properties near the dam site to evaluate materials for potential use as borrow in earthen embankment construction. The geotechnical investigation was carried out by Braun Intertec. A description of the geotechnical investigation program is provided in Section 2.2.

A copy of the preliminary Braun Intertec Geotechnical Investigation Report is included in Appendix C. The report describes methods used to advance borings and collect and test soil and bedrock samples. The report includes boring logs and laboratory test results.

The results of the geotechnical investigation indicate variable foundation conditions along the dam alignment. A profile sketch of the foundation is included in Appendix C. A description of subsurface conditions encountered is provided below based on existing structure locations:

- **North Embankment** – subsurface consists of up to 28 feet of sand and gravel fill material, underlain by approximately 10 feet of sandy lean clay. The sandy lean clay is underlain by approximately 15 feet of poorly graded sand to approximately elevation 852, where limestone bedrock is encountered.
- **Existing Powerhouse and Spillway** – concrete rests atop limestone bedrock, which is encountered at approximately elevation 848.
- **Dam Breach** – subsurface consists of a varied depth of sand and gravel underlain by limestone bedrock. The bedrock elevation drops off sharply moving from north to south. Bedrock is encountered at elevation 861 at boring location ST-4, elevation 842 at boring location ST-5, and bedrock is not encountered at boring location ST-6 (to elevation 817).
- **South Abutment** – subsurface consists of approximately 20 feet of sandy lean clay fill, underlain by poorly graded sand extending to the limits of the borings at 70 feet in depth. Bedrock was not encountered.

The borings taken at potential borrow sites typically encountered two soil types underlying the topsoil: a silty clay loess overlying a silty clay glacial till soil. The loess soils, while potentially acceptable for embankment construction, were typically encountered at very high moisture contents, requiring excavation and spreading (farming) in order to get the material to an acceptable moisture content for placement and compaction. The till soils typically provide a superior material for embankment construction and have in-situ moisture contents closer to those required for placement and compaction in an earthen embankment. The till soils were encountered at depths of 12 feet or more, under the loess soils, so significant excavation would be required to develop these soils for borrow. Additional future investigations by the Contractor may locate the till soils at shallower depths for borrow development. Both materials indicate acceptable strength and seepage properties for use in earthen dams.

Embankment Seepage Analysis

Seepage analysis was conducted for proposed embankment and seepage control measures using GeoStudio's SEEP/W finite element seepage modeling program. Soil classification and laboratory gradation results were used to develop input seepage parameters. Permeability coefficients were determined according to Hazen's empirical formula using D_{10} values (particle diameter corresponding to 10% passing). The proposed service and auxiliary embankments (located within the current breach) were modeled with various cutoff depths and configurations. Horizontal blanket drains were also included in the model, for safe collection and conveyance of seepage flows, without saturating the downstream slope of the

embankments. Exit gradients (exit gradient is defined as the rate of change of total head pressure with distance) and seepage flow rates were analyzed to come up with an optimized and adequate cutoff/drainage system. To achieve the target factor of safety of 1.5, the target exit gradient was assumed as 0.67. This assumes a critical gradient of the material of 1.0. To achieve the target gradient, the sheet pile cutoff was designed as 35 feet below base of the new embankment (into sand foundation). The existing south embankment was also analyzed for seepage, to determine required depth of seepage cut-off beneath this shorter embankment section. Due to uncertainties with the condition and construction of the existing cut-off and embankment, it is conservatively omitted from the analysis. For the existing south embankment, the analyses indicate a 40-foot cut-off is required (20 ft through existing clay embankment and 20 ft into sand foundation).

Soil boring ST-4 encountered poor rock quality and large voids near the north end of the new proposed spillway embankment. To provide a positive seepage cutoff between the new embankment sheet pile cutoff and the steep bedrock slope, a grouting program will be required at this location.

Embankment Stability Analysis

Stability analyses were completed to determine required slopes and footprint of the proposed embankment alternatives so that current dam safety design guidelines are met. Stability analysis was carried out using GeoStudio's SLOPE/W (2007) modeling program. Spencer's Method was used to find minimum factors of safety for various loading conditions. The analyses were completed in general conformance with the requirements for new earth and rock-fill dams presented in the USACE Slope Stability Engineering Manual (EM 1110-2-1920). Table 3-1 summarizes load conditions and required factors of safety.

Table 3-1 Slope Stability Requirements

Load Condition	Required FOS
Total Stress	1.3
Effective Stress	1.5

A maximum surcharge pool was assumed with water to the top of the proposed spillway crest. To account for the decreased water surcharge loading as a result of the labyrinth weir, 50% of the water surcharge load was considered along the width of the new spillway. A rapid drawdown condition was not modeled at this stage in the design because it is unlikely that the pool will ever be rapidly drained.

For proposed new embankment sections, slope stability was analyzed for embankments constructed of locally available borrow materials (identified in Braun Intertec investigation) as well as roller compacted concrete (RCC). It was determined that 3 horizontal on 1 vertical slopes are required for the both the upstream and the downstream faces of embankments constructed of loess or till in order to satisfy all design requirements. Roller compacted concrete faced embankments meet design requirements if constructed with 2.5 horizontal on 1 vertical downstream slopes.

Settlement Analysis

Long-term consolidation settlement is not anticipated as embankment construction will take place on subsurface sands. Given the sand foundation material, a majority of settlement will occur as construction proceeds. Settlement within the embankment fill will be limited by proper placement, moisture control, and compaction of embankment fill.

3.3 Structural

Existing Spillway and Powerhouse Stability

A review of the 1997 Ashton-Barnes stability analysis of the dam indicated several deficiencies when compared to current dam safety design requirements (Discussed in Section 2). As a result, Stanley Consultants conducted a new stability analysis of the ogee spillway and powerhouse structures for sliding and overturning stability in general conformance with the requirements of the USACE *Design of Gravity Dams* (EM 1110-2-2200) and the FERC *Engineering Guidelines for the Evaluation of Hydropower Projects*, Chapter 3. The analyses evaluated the stability of the concrete structures as constructed and investigated options for anchoring the structures to the foundation bedrock in order to meet current dam design criteria. These criteria included:

1. Assuming the dam is a moderate hazard classification (see discussion in Section 3.3) and using applicable FERC structural criteria and design flood headwater and tailwater conditions.
2. The existing ogee spillway and powerhouse structures were checked against both USACE and FERC stability criteria. Rock anchor alternatives were designed to meet either USACE or FERC requirements. In the analysis, it was found that, for these two structures, FERC requirements were more stringent than the USACE. The reason for designing anchors to FERC standards is that, if hydropower generation at the dam is ever rehabilitated, there would be cost savings in adding the additional anchors at this time, versus adding at a later date.
3. Headwater and tailwater elevations reflect the latest hydrology and hydraulic modeling results.
4. A new geotechnical investigation was conducted and foundation parameters were based on new test information and research on similar bedrock founded projects.

Concrete core borings were advanced by Braun Intertec through a powerhouse wall and a central pier of the ogee spillway for the purpose of evaluating bedrock conditions underlying the two structures as well as the potential for concrete to bedrock bond at this interface. The results of the core borings and laboratory testing indicate that the bedrock is of sufficient quality to support the structures and to develop required capacity of future rock anchors. The core borings also indicated a clean interface between concrete and bedrock and that some bonding of concrete to bedrock exists at this interface.

Table 3-2 shows the criteria used in the sliding stability analysis for the existing spillway and powerhouse structures.

Table 3-2 Stability Parameters

Load Condition	USACE Minimum Sliding FOS	FERC Minimum Sliding FOS (Cohesion not Used)
Usual	2.0	1.5
Unusual	1.7	1.5
Extreme/Post Earthquake	1.3	1.5

To satisfy USACE's sliding factors of safety, ten (10) rock anchors are required to stabilize the existing ogee spillway structure, and ten (10) anchors are need for the powerhouse. In order to meet FERC's criteria, thirty (30) would be required for the spillway and twenty (20) for the powerhouse.

Detailed parameters and assumptions used in the analysis are presented in Appendix D.

Repair of Existing Structures

General Concrete Condition

During Stanley Consultants' September 2010 inspection and later site visits, concrete deterioration and spalling were observed in many areas of the powerhouse and spillway structures. Concrete on the surface of the spillway and lower portion of the bridge piers was found severely deteriorated. Concrete around the lift gate slots spalled to prevent a gate to open. Based on the surficial observations, a concrete coring and testing program was developed to evaluate the subsurface condition of the concrete. The coring information and test results are provided in Appendix C.

Based on these results, it is reasonable to assume that the concrete below the spillway and pier/wall surface has acceptable strength. By removing and replacing the deteriorated surface no further structural evaluation of the concrete should be required. If the condition of concrete found during construction differs significantly from the concrete coring and testing results, a further evaluation of the structure will be conducted.

Powerhouse

Stanley Consultants analyzed and designed modifications to the powerhouse assuming the structure acted as one monolith so the powerhouse essentially functions as a water retaining structure. It was assumed that structural upgrades needed for hydropower generation would be completed at a later stage should the facility be restored for generating electricity. Therefore, in this phase of the project, in addition to anchoring the structure to bedrock foundation to meet USACE and/or FERC stability requirements, structural repair work was limited to the portions that were deemed necessary for the powerhouse and spillway to function as a water retaining structure.

During the site visit on November 9, 2010, significant evidence of seepage through the powerhouse roof was observed. There was also evidence of potential corrosion of the reinforcing steel by staining observed along cracks in the ceiling. It is Stanley Consultants' understanding that the County (with assistance from Iowa State University)

completed load testing of the bridge deck in response to observed conditions and results of the testing revealed no structural deficiencies. Waterproofing of the roof slab is proposed to minimize further infiltration and degradation. Subsequent use of the bridge deck will be limited to construction and maintenance equipment. However, given the large size and weight of construction vehicles and loads, the bridge will be further analyzed during detailed design when equipment and material weights are known to determine if vehicle restrictions will be required during construction. To minimize further degradation of the bridge deck reinforcement the use of de-icing solutions should be discontinued.

North Downstream Abutment Wall

The masonry block portion of the north downstream abutment wall was observed to be in poor condition. The concrete wall that the blocks were founded on appeared to be in satisfactory condition. Reconstruction of the stone block retaining wall portion of this wall is proposed.

Spillway

The existing ogee spillway will be anchored to bedrock foundation to meet USACE and/or FERC stability requirements. Deteriorated concrete on the ogee sections and piers will be removed and replaced with new concrete. All gate slots will be repaired or replaced to accommodate the new gate system proposed. The bridge structure over the spillway section is relatively new, no significant deterioration was observed during inspections, and the bridge will no longer be utilized for public access, therefore, no significant structural repair work was proposed for the bridge.

Damaged or deteriorated concrete at the spillway training walls and stilling basin will be repaired. The remnants of the existing fish ladder at ogee spillway south training wall will be removed. The training wall will be repaired and modified to accommodate the new spillway structure to be located on the south side of the wall.

Construction of New Spillways

New spillway alternatives were designed to pass 100-year design flood, and have an overall capacity to pass ½ PMF flood.

Construction of new structures, including spillway weir, spillway slab, stilling basin, retaining/training walls, will meet both USACE and FERC requirements for stability and structural strength.

The conceptual structural designs were based upon the following:

Cast-in-Place Concrete Design:

- Conform to “Building Code Requirements for Reinforced Concrete,” ACI 318-08.
- f_c = 4,000 psi for all structural concrete.
- Reinforcing Steel: ASTM A615/A615M, Grade 60.

Structural Steel Design:

- Conform to latest edition of AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings."
- F_y = 36 ksi (yield point), based on using steel conforming to ASTM A36.
- F_y = 50 ksi (yield point), based on using steel conforming to ASTM A572 for sheet-pile.

Loads:

- All loads per ASCE 7-05 and IBC-2006.

Materials and Construction:

- Concrete:
  - Specify that average 28-day compressive strength shall exceed f_c on basis of standard deviation, in accordance with ACI procedures.
  - Use air-entrained concrete for all structures.
  - Allow fly ash in all concrete.
  - Use ASTM C150, Type I cement for all concrete.
- Reinforcing Steel:
  - Deformed billet steel, ASTM A615, Grade 60.
  - Wire fabric, ASTM A185.
- Structural metals:
  - Grade: ASTM A36.
  - ASTM A572, grade 50 for sheet-pile.
  - Protect ferrous metals from corrosion.

Design criteria and parameters used in design are included in Appendix D.

Seismic analysis for the structures is not necessary, since the dam is located in a low seismic zone. S_s = 0.086, S_1 = 0.046.

3.4 Hydrology/Hydraulics

Lake Delhi Dam in its pre-failure condition did not have sufficient hydraulic capacity to pass the new project design flood for a Moderate Hazard Dam. Design of the reconstruction will include significantly increasing Lake Delhi Dam's hydraulic capacity for passing flood flows.

For the alternatives analysis several concepts were developed for reconstructing the dam's spillway(s). Three concepts were taken to preliminary design and evaluated for potential design and construction. Other hydraulic considerations included minimum/low flow passage, lake draining capacity, and cofferdam/bypass during construction. Steps to complete the hydrologic and hydraulic studies for the alternatives analysis included:

- Characterizing Maquoketa River Flows at Lake Delhi Dam.
- Developing a hydrologic model of Lake Delhi Dam watershed.
- Developing a hydraulic model of Maquoketa River upstream and downstream of Lake Delhi Dam.
- Performing hazard classification and design flood analysis for Lake Delhi Dam.
- Developing Lake Delhi Dam spillway concepts.
- Addressing other hydraulic issues.

Maquoketa River Flows

U.S. Geological Survey (USGS) maintains a river flow gage near Manchester, Iowa at the Highway 20 crossing roughly 12 miles upstream of Lake Delhi Dam. USGS performed a frequency analysis of gage flows which were adjusted by USGS regional drainage area ratio methodology to estimate return period flows at Lake Delhi Dam. Results are provided in Table 3-1.

Table 3-3 Return Period Flows

Return Period (yrs)	Annual Exceedance Probability	USGS Gage Flow (cfs)	Lake Delhi Dam Flow (cfs)
1	0.95	1,393	1,491
2	0.5	4,506	4,821
5	0.2	8,636	9,241
10	0.1	12,300	13,161
25	0.04	18,130	19,399
50	0.02	23,420	25,059
100	0.01	29,610	31,683
200	0.005	36,820	39,397
500	0.002	48,150	51,521

Average daily flows at Lake Delhi Dam are in the range of 150 cubic feet per second (cfs).

Hydrologic Model

A HEC-HMS hydrologic model was used to develop a series of design flood hydrographs (i.e. analysis derived) for the Lake Delhi Dam watershed. The flood hydrographs were used as an input for the hydraulic model.

The probable maximum flood (PMF) hydrograph was developed using ArcGIS to establish watershed parameters and NOAA's HMR 51/52 publication to establish rainfall depth-durations. The full and ½ PMF were used in the analysis. The 100-year flood hydrograph was developed using the same ArcGIS watershed parameters and the 100-year/24-hour rainfall was obtained from *Iowa Rainfall Frequencies*.

The peak HEC-HMS derived 100-year flow was checked against the peak 100-year flow established at the USGS streamflow gage at Manchester and the two flows matched closely. Watershed parameters, rainfall depths, and peak flood flows are provided in Table 3-2.

Table 3-4 Lake Delhi Dam Watershed Parameters

Parameter	Value
Drainage Area (mi^2)	349
Infiltration (in/hr)	0.25
Time of Concentration (hrs)	18
Storage Coefficient (hrs)	15
PMF Rainfall Total (in)	25.8
PMF Peak Flow (cfs)	143,900
100-Year Rainfall Total (in)	6.4
100-Year Peak Flow (cfs)	28,100

Hydraulic Model

The starting point for the hydraulic modeling was the HEC-RAS model of the Maquoketa River developed by the DNR to evaluate the 2010 breach of Lake Delhi Dam. The upstream end of the river model is at the Highway 20 Bridge and the model extends approximately 23 miles to just downstream of Hopkinton.

The HEC-RAS model as well as supporting background data was provided to Stanley Consultants by the DNR. The following adjustments were made to the DNR HEC-RAS model:

- River channel topography was updated with post-breach LiDAR data obtained in 2010.
- Bridge structures were added downstream of the dam (Quarter Road., 295th Street and Hopkinton).
- One inflow hydrograph was used at the upstream end of the model (DNR model used two).
- The dam was modified to reflect the proposed condition (working gates, principal/auxiliary spillway.

Hazard Classification

Hydrologic and Hydraulic analysis and design standards for dams in Iowa are specified in *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams*. The standards are defined according to the dam's hazard classification. The state of Iowa has three hazard classifications for dams; Low, Moderate, and High Hazard.

If hydropower is ever redeveloped at Lake Delhi Dam, the reconstructed dam will have to meet FERC criteria. FERC also has three hazard classifications; Low, Significant, and High

Hazard. The FERC and DNR hazard classification definitions are very similar so the classification determined by DNR criteria should correspond to a FERC hazard classification. Table 3-3 provides the agency hazard classification definitions.

Table 3-5 Hazard Classification Definitions

Hazard Class	DNR Definition	FERC Definition
Low	Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm outbuildings, agricultural lands, and lesser-used roads, and where loss of human life is considered unlikely.	Structures located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.
Moderate/ Significant	Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.	Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.
High	Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.	Structures located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

The hazard classification of Lake Delhi Dam controls several design parameters including the freeboard design flood. For detailed design to proceed, a hazard classification is needed to establish the applicable dam safety and design criteria.

Previous inspections and analyses have identified Lake Delhi Dam as a low, moderate, and high hazard structure, but there has not been a detailed analysis of potential downstream hazard to substantiate the hazard classification. The hazard classification analysis performed for this project provides a more thorough evaluation of risk associated with theoretical dam failure through inundation mapping of a series of flood events with and without dam failure.

The full PMF, ½ PMF, and 100-year flood were modeled in HEC-RAS with and without dam failure (breach). The DNR established dam breach parameters for their original HEC-RAS model based upon the Lake Delhi Dam failure observed in 2010. For the reconstructed dam analysis, the width of the dam breach was reduced from 250 feet to 175 feet to better reflect the reconstructed condition. The breach formation time was left at 1.5 hours. The failure was set to initiate at the peak of the flood hydrograph which yields the highest flood elevation (i.e. worst-case condition).

Failure of the existing powerhouse and gated spillways were also evaluated but the embankment failure provided the most critical dam failure scenario.

Design of the Lake Delhi Dam reconstruction is in the preliminary stage, so the "reconstructed" dam in the HEC-RAS model represents an approximation. Gates will be replaced as part of the reconstruction so they were assumed to be fully operable in the HEC-RAS model with the same opening area as the existing condition.

A representative principal/auxiliary spillway was added to the HEC-RAS model. Hazard classification is focused more on the downstream impact of the dam than the specifics of the spillway so using a principal/auxiliary spillway approximation is reasonable for this analysis. The various spillway options currently being considered have a similar embankment shape so the proposed HEC-RAS model should provide an adequate depiction of the failure condition no matter which alternative is chosen. However, the analysis will be updated once the reconstruction design is established, but a significant change in results is not expected.

The HEC-RAS flood profiles were exported to ArcGIS using HEC-GeoRAS, which uses the profiles to develop inundation extents for each flood/failure event. Inundation maps were created that include geo-referenced aerial imagery so the inundation limits can be viewed relative to downstream buildings and infrastructure. Detailed inundation maps and tables are provided in Appendix B. Results are summarized in Table 3-4.

Table 3-6 Impacted Structure Summary

Event	Scenario	Residential	Comm/Ag	Bridges	Roads
PMF	No Breach	104	30	3	12
	Breach	107	30	3	12
Half PMF	No Breach	27	8	3	8
	Breach	29	8	3	8
100-YR	No Breach	3	1	1	5
	Breach	5	2	1	5
Sunny Day	Breach	0	0	0	1

Hazard classification is based on the potential consequence of dam failure. When analyzing the consequences of dam failure during a flood event it is the increase in consequence (i.e. increase in damage and potential loss of life) due to failure that is evaluated.

Results of the HEC-RAS modeling and inundation mapping indicate that dam failure during flood events does not appear to cause a significant increase in the number of structures inundated. The majority of additional structures that are inundated by a failure event are the homes within 1500 feet downstream of the dam. As the Emergency Action Plan is developed for the reconstructed condition it will be important to have well-defined communication and evacuation procedures defined for these residents.

Hazard classification was reviewed for both the DNR and FERC definitions. Lake Delhi Dam appears to fit the Moderate (DNR), Significant (FERC) Hazard Classification. The reasoning is as follows:

- HEC-RAS modeling and inundation mapping show that a potential failure during a flood would only cause a small increase in the number of structures impacted.
- A potential sunny day failure conditions stays within the limits of the 100-year floodplain (typically non-developed area) so the potential for damage is less than if sunny day failure flooded more habitable or developable lands.
- Much of the area downstream of Lake Delhi Dam is rural and agricultural. Although future development is possible, most development would likely occur closer to the town of Delhi, which is up above the river channel or in Hopkinton which is far enough downstream that the increase in flood elevation due to failure is roughly 1 foot.
- The Maquoketa River downstream of Lake Delhi Dam is widely used for canoeing and fishing activities, however the river does not contain the type of attractions that bring large numbers of people into the river channel area for extended periods of time (i.e. restaurants, resorts, large campgrounds or trailer parks, etc.)
- Therefore, the DNR definition of Moderate hazard where "…failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life." is appropriate for the reconstructed Lake Delhi Dam.
- The FERC definition of Significant hazard for "Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways, or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life." also seems the appropriate classification for Lake Delhi Dam.

Design Flood

Per *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams*, a moderate hazard classification establishes the freeboard design flood as the ½ PMF. FERC uses an incremental analysis to establish the design flood by determining the largest food where failure causes an increase in downstream hazard. An incremental analysis was performed and using the DNR designated ½ PMF as the freeboard design flood should also meet FERC criteria.

Recommendation for Final Design

Based on the analysis Stanley Consultants recommends that design of the Lake Delhi Dam reconstruction proceed with a classification as a Moderate Hazard structure and a freeboard design flood of the ½ PMF. This classification will be verified with an updated analysis once reconstruction design has been established.

A detailed hydrologic and hydraulic studies report, computations are included in Appendix B.

Spillway Concepts

Using the ½ PMF as the design flood, spillway concepts were developed with the objective of the reconstructed Lake Delhi Dam being able to pass the ½ PMF without overtopping the existing powerhouse/gated spillway structure.

Prior to the breach, flood flows were passed by opening the three spillway gates located adjacent to the powerhouse. The gate system has a hydraulic capacity of roughly 30,000 cfs with the gates fully raised and the upstream pool at the top of dam. Reconstructed Lake Delhi Dam will need to pass roughly 69,000 cfs which is more than double the hydraulic capacity of the pre-breach dam.

The new spillway system at Lake Delhi Dam will need to provide roughly 39,000 cfs of additional hydraulic capacity. In performing preliminary design of the spillway alternatives, stage-discharge curves were developed for each spillway alternative.

For the labyrinth weir spillway alternatives a set of empirical equations was used to develop stage-discharge curves. Labyrinth weir hydraulics has been studied in detail so it is possible to predict the discharge rating for a given geometry with reasonable accuracy. *Hydraulic Design of Labyrinth Weirs* was utilized for developing the geometry and estimating the discharge capacity of the labyrinth weir alternatives.

The pneumatic gate spillway alternative essentially acts as a sharp crested weir with an adjustable crest. When flows are low, the crest is kept at or near the normal pool and as flows increased the gate panels are lowered until they are flush with the fixed concrete slab/crest they are mounted to. For preliminary design, the controlling factor is passage of the design flood, so gates were assumed to be down with the weir crest elevation essentially at the fixed concrete slab/crest and stage-discharge curves were computed.

A description of the alternatives analysis is provided in Section 4. A detailed description of hydraulic design and analysis is provided in Appendix B.

Minimum/Low Flow Passage

Minimum/low flow passage was a topic of concern with operation of the pre-breach Lake Delhi Dam. During times of normal and low flows, flow downstream of the dam was controlled by wicket gate discharge. Wicket gate settings and pool elevations were recorded but discharge rates were not quantified. During times of low flow there were concerns that insufficient discharge was being provided to the downstream waterway.

An additional concern was dissolved oxygen levels of the discharge. The wicket gates intake elevation is at 881.3, roughly 15 feet below the normal pool elevation where dissolved oxygen levels are typically low. Discharge through the gates was not aerated so waters in immediate downstream channel frequently did not meet dissolved oxygen requirements.

If the wicket gates are restored as the normal means of discharge, an aeration mechanism will be incorporated into the system. If the labyrinth or pneumatic gates are used as the single principal spillway sufficient aeration will be provided by the pool level discharge and flow down the spillway chute.

In addition to the spillway alternatives, installation of valved openings in two of the new lift gates is being considered. During normal operating conditions the valves would be closed. However the valves could be used to:

- Provide additional discharge capacity prior to gates lifting (roughly 150 cfs for two 30-inch valves at normal pool).
- Provide minimum flow passage if the upstream pool drops below the principal spillway crest.
- Provide bypass flow during potential maintenance work or debris removal at the principal spillway without lifting gates.
- Provide the capability to draw down the pool a small amount or maintain a slightly drawn down pool during low flows. The lift gates are good for passing large flows but not for normal bypass flows or drawing down the pool a few inches.

Unlike the wicket gates, the valves will discharge onto the concrete ogee spillway, so even though the valves would likely be 10 feet below the normal pool, discharge would be aerated by the drop over the concrete spillway.

The previous dam operator indicated the 7Q10 flow (lowest seven-day average flow that occurs once every 10 years) for the Maquoketa River at Lake Delhi Dam is roughly 28 cfs. The 30-inch valves would have the capacity to discharge the 7Q10 flow.

As reconstruction design progresses a detailed operating manual will be developed with DNR input and approval that provides operating protocol and discharge rates for the expected range of flow conditions.

Lake Draining Capacity

DNR requires that "A gated low level outlet shall be provided which is capable of draining at least 50 percent of the permanent storage behind the dam within a reasonable length of time." The existing lift gates provide sufficient capacity to drain 50 percent of the volume below the normal pool elevation. In addition, existing plans indicate a set of two 37.5-inch diameter sluice pipes were installed through the northernmost spillway pier approximately 20 feet below the crest of the gated spillway.

If they do exist, the sluice pipe intakes are buried under 20 feet of riprap. This riprap will be removed during the dam reconstruction and the feasibility of restoring the existing sluice pipes will be assessed. The sluice pipes are not necessary to meet DNR design requirements but could be useful during construction and for future maintenance and dredging projects.

Section 4

Reconstruction Alternatives Development/Evaluation

4.1 General

The reconstruction project required to restore the Lake Delhi Pool and to bring the facility into compliance with current dam safety and design criteria will require repair work on all of the existing project features described in Section 1. In addition, construction or installation of new features will be required to enhance the safety and performance of the facility. Some of the features are limited to a single option, with no cost effective or practical alternatives available. These features/repairs are called Reconstruction Non-Alternatives and are discussed in Section 5. Other features have one or more alternatives that have enough merit to warrant a preliminary design and cost evaluation to determine the optimum alternative that best meets the District's project objectives. These features are described in this section.

On November 9, 2011, a multi-disciplined team of Stanley Consultants engineers completed a site visit to collect additional data on the equipment and construction of the existing project features, as well as their current condition. Members of the team represented the Civil, Hydrology/Hydraulics, Geotechnical, Structural, Electrical, and Mechanical engineering disciplines. Following the site visit, the Stanley Consultants team met with the District Trustees for an Alternatives Development "Brain Storming" session. The purpose of the session was to:

- Establish District Objectives.
- Review parameters for design development and alternative evaluation.
- Initiate the creative "brain storming" process for alternative development.

The District's Project Objectives were used as the criteria for alternative development and evaluation. Objectives for the reconstruction project include the following:

- Meet requirements of current dam safety and design standards.
- Minimize operation/maintenance requirements.
- Maintain or improve upstream and downstream flow conditions.
- Provide adequate (50+ year) service life.
- Increase public safety at dam site.
- Improve public recreational opportunities.
- Reduce potential for damage from debris flow.
- Provide cost-effective solution.
- Constructability.
- Minimize right-of-way impacts.
- Minimize permitting requirements.
- Provide opportunity for greater pool control.
- Enhance fisheries opportunities.
- Improve water quality.

The Alternatives Kick-Off Meeting provided the Stanley Consultants design team with an understanding of the District's objectives for the reconstruction and performance of the project. Working with the District, Stanley Consultants developed a list of potential alternatives for the repair of existing features and construction of new features. During the alternative concept design and evaluation process, it became apparent that some alternatives were unsuitable due to excessive cost and/or failure to sufficiently meet one or more District's Objectives. Conceptual design and cost estimating was not completed for unsuitable alternatives. The evaluation process for each set of alternatives is described in this section.

4.2 North Embankment

The north embankment consists of the portion of the dam extending north of the powerhouse structure and tying into the north river bank. The existing upstream and downstream walls are showing signs of deterioration, damage, cracking, etc. and will be removed. Available drawings indicate that there is a third concrete wall located within the embankment that was likely the upstream wall prior to the construction of the crib wall and widening of the approach to the dam bridge. This wall will also be removed as part of preparation for the new structure. Site configuration and right of way limits at this location eliminate construction of a full earthen embankment as an alternative at this location. Three separate structural alternatives were considered for reconstruction of the north embankment:

- Double Sheet Pile Wall.
- Cellular Sheet Pile Structure.
- Reinforced Concrete Walls.

Each alternative includes an upstream sheet pile seepage cutoff driven to bedrock. Each alternative also maintains an approximately 25-foot wide roadway atop the embankment. The roadway section will require installation of a guardrail or parapet wall to contain traffic.

The double sheet pile wall alternative consists of two rows of Z-section sheet pile wall driven parallel to one another and tied together with anchor rod. The upstream wall sheet pile will be driven to bedrock to serve as a seepage cutoff. Advantages of the double sheet pile wall include: low-cost alternative, basic construction methods and no temperature restrictions for sheet pile installation. Disadvantages include a non-aesthetic wall face and anchor ties below the upstream water surface would be difficult to inspect.

The cellular sheet pile structure alternative consists of PS-section sheet pile driven in circular "cell" configurations with connector arcs. The upstream sheet pile will be driven to bedrock to serve as a seepage cutoff. Advantages of the cellular sheet pile structure include: single construction operation and no temperature restrictions for sheet pile installation. Disadvantages include a non-aesthetic wall face, high cost of steel sheet pile, and templates required for construction.

The reinforced concrete walls alternative consists of a U-shaped concrete walls/footing structure with anchor ties between wall stems. In addition to the reinforced concrete, an upstream sheet pile will be driven to bedrock to serve as a seepage cutoff. Advantages of the reinforced concrete structure include: options for an aesthetic wall face. Disadvantages include multiple construction operations and temperature restrictions for concrete work.

Given the cost comparison and aesthetics potential, the recommended alternative for the north embankment is the reinforced concrete wall.

Conceptual drawings of the North Embankment Alternatives are provided in Exhibits 2-4 in Appendix F.

(Note: All costs shown in the comparison tables in this section have been adjusted to include markups for contingency and inflation.)

Table 4-1 North Embankment Alternative Cost Comparison

Alternative	Cost
Double Sheet Pile Wall	$469,000
Cellular Sheet Pile Structure	$675,000
Reinforced Concrete Walls	$536,000

4.3 North Downstream Abutment Wall

The North Downstream Abutment Wall extends downstream from the Powerhouse Structure. The base and lower portion of the wall is reinforced concrete and the upper portion is masonry block. The existing masonry block portion of the wall is showing signs of deterioration/damage and replacement is recommended. The following alternatives were considered:

- Leave the existing wall as-is.
- Remove and replace the masonry block portion of the wall with large block or mechanically stabilized earth (MSE) wall.
- Remove and replace the masonry block portion of the wall with a reinforced concrete wall.

Leaving the existing wall in place is not recommended because the existing masonry block wall is showing signs of instability and severe cracking. Even though the masonry block wall's failure would not likely compromise the stability of the concrete portion of the wall or the powerhouse structure its failure could impact access to the downstream entrance to the powerhouse.

The reinforced concrete wall alternative would consist of removing the existing block wall and reconstructing a reinforced concrete wall. The advantages of this alternative include matching construction of adjacent concrete walls and the use of in-place fill soils. Disadvantages include the required excavation to frost depth and temperature restrictions of placing concrete.

The MSE wall alternative would consist of removing the existing wall and reconstructing a modular block wall in its place. The advantages of this alternative include replication of previous construction, aesthetic wall face, less temperature restriction during construction, and the block wall is better-suited to endure any freeze/thaw movement or settlement which may occur. Disadvantages include a requirement for engineered fill which may need to be imported.

Reconstruction of the masonry block portion of the wall would also include removal of the elevated concrete slab at downstream face of the powerhouse and filling of the void. The new wall would extend to the face of the downstream powerhouse wall.

Given the cost comparison and overall aesthetics, the recommended alternative for the north downstream abutment wall is the MSE wall.

Table 4-2 North Downstream Abutment Wall Alternative Cost Comparison

Alternative	Cost
MSE Wall	$106,000
Reinforced Concrete Wall	$181,000

4.4 Powerhouse

Several rehabilitations, replacements, and improvements to the powerhouse structure are being recommended as part of the dam reconstruction project. The major work item is enhancing powerhouse stability to meet current dam safety and design standards. For the alternatives study, both FERC and USACE stability standards were evaluated. The major differences between the two agencies' stability standards include:

- For the sliding stability factor of safety, generally FERC requirements are more stringent than USACE, if the same parameters are used in analysis, i.e. both cohesive bond and sliding friction assumed at the dam foundation interface.

- When cohesive bond at the dam foundation interface cannot be verified by borings or tests, FERC recommends an alternative minimum factor of safety be used in conjunction with a no cohesion assumption. A minimum factor of safety of 1.5 is required for all static load cases.
- For dam stability regarding overturning and foundation bearing pressure, FERC criteria closely resemble the criteria used by USACE.

Table 4-3 Stability Parameters

Load Condition	USACE Minimum Sliding FOS	FERC Minimum Sliding FOS
Usual	2.0	3.0
Unusual	1.7	2.0
Extreme/Post Earthquake	1.3	1.3

To meet the dam design standards of either agency, the stability of the existing powerhouse must be improved to satisfy the structure sliding factor of safety under the design flood condition. The proposed method to increase the stability of the powerhouse structure is installation of post-tensioned rock anchors through the base of the powerhouse and into the underlying bedrock.

Two alternatives were developed: one to satisfy USACE's dam safety criteria, the other is to meet FERC's dam safety requirements. FERC standards would be required should hydropower ever be redeveloped at the dam. USACE standards are considered sufficient for non-hydro generating dams. Installation of the rock anchors during the reconstruction project would be significantly less expensive than mobilizing a contractor to install additional anchors at a later date. The powerhouse structure will need approximately ten (10) rock anchors in order to meet USACE stability requirements. These anchors would be located at the upstream face of the powerhouse. Excavation to bedrock (including removal of upstream riprap) will be required for installation of the anchors. A concrete "bench" would be constructed upstream of the powerhouse and doweled into the powerhouse to provide a location for rock anchor installation.

Meeting FERC criteria will require installation of twenty (20) rock anchors. These anchors will require higher capacity, due to increased load requirements and limited accessibility for installation. Ten (10) anchors will be installed at the upstream face of the powerhouse, and the other ten (10) will be installed through the solid concrete walls of the powerhouse.

The recommended alternative for the powerhouse is to anchor the structure to meet USACE dam design standards. With the future of hydropower development being uncertain and given the significant additional cost, meeting FERC criteria is not recommended.

Conceptual sections of the Powerhouse stabilization are provided in Exhibit 8 in Appendix F.

There are two options for waterproofing the powerhouse roof bridge deck. One alternative would be to clean the deck and epoxy seal any visible cracks in the concrete. The second alternative involves installation of a waterproofing membrane system with asphaltic concrete deck overlay.

The membrane system is recommended for the powerhouse roof rehabilitation since it will provide longer-lasting, more extensive water protection.

Table 4-4 Powerhouse Alternative Cost Comparison

Stabilization Alternatives	Cost
USACE Criteria	$287,000
FERC Criteria	$639,000

Waterproof Alternatives	Cost
Clean Deck/Epoxy Seal	$21,000
Waterproof Membrane System	$37,000

4.5 Existing Spillway

Rehabilitation of the existing gated ogee spillway includes these major items:

- Anchoring the dam structure to the bedrock foundation in order to meet current dam design standards – either USACE or FERC.
- Lift gate repair or replacement.

Similar to the powerhouse, there are two options for increasing the stability of the existing spillway.

To anchor the spillway to meet USACE criteria: approximately ten (10) rock anchors are required. The anchors would be installed either in front of spillway upstream face, or through the spillway crest. The first option would require excavation at upstream face of the spillway to the base of the dam and a new concrete bench doweled into the existing structure. This option would provide relatively easy access for construction. The second option would require drilling anchor holes through existing concrete approximately 30 feet thick, and accessibility for construction would be more difficult.

To anchor the spillway to meet FERC criteria, approximately thirty (30) rock anchors would be required. Twenty (20) of the anchors would be installed as described for the two options above. USACE and an additional ten (10) anchors would be located in the spillway piers.

The recommended alternative for the spillway stabilization is to anchor the structure to meet USACE dam design standards. With the future of hydropower development being uncertain and given the significant additional cost, meeting FERC criteria is not recommended.

A conceptual section of the spillway structure stabilization is provided in Exhibit 8 in Appendix F.

Several gate options were considered for the dam reconstruction; however, the pier and bridge configuration above the spillway is not conducive to different gate systems. The options considered are shown in Table 4-5.

Table 4-5 Spillway Gate System Comparison

Option	Suitable	Explanation
Radial Gates	No	Radial gates are mounted on an arm and are lifted by rotating the arm upwards so have a circular motion. Installing radial gates at the existing spillway would require removing a significant portion of the bridge deck.
Crest Gates	No	Crest gates are mounted to the crest of the spillway and when lowered are flush with the crest. The ogee spillway at Lake Delhi Dam is steep and does not have a wide crest, so installation of crest gates would require removal of large portion of the crest to create a platform for mounting the gates.
Lift Gates	Yes	The existing spillway used lift gates so the configuration is suitable for lift gate installation. The gate guides were damaged and need replacement but that repair would be minor compared to the work required to install other gate systems.

Prior to the 2010 dam failure a project was underway to replace the lift gate hoisting mechanism. The hoisting equipment was received by the dam operator but never installed at the dam so could be installed as part of the reconstruction project. The new hoisting equipment should eliminate previous issues experienced with lifting gates and provides an additional 3 feet of lifting height, so the new gate openings will be 25 feet wide by 20 feet high when the gates are fully lifted.

The recommended alternative is to replace the existing lift gates. Since the structure was originally configured to lift gate operation, there are less modifications required compared to the other gate system alternatives. While replacement of the existing lift gates will involve some structural updates (replacement of the slide inserts, new actuators, etc), the basic structural elements are in place. In addition the hoisting mechanism received for the 2009 upgrade project that was never installed can be installed and used with new lift gates.

Table 4-6 Existing Spillway Alternative Cost Comparison

Spillway Anchoring Alternative	Cost
USACE Criteria	$324,000
FERC Criteria	$607,000
Spillway Gate System Alternative	**Cost**
Replace Lift Gate System	$2,044,000

4.6 New Spillway

The new spillway system at Lake Delhi Dam will need to provide roughly 39,000 cfs of additional hydraulic capacity for the dam to pass the design flood of ½ PMF without overtopping

the powerhouse or spillway gate structure. There is roughly 230 feet between the buttress wall at the southern end of the existing powerhouse/spillway structure and the southern riverbank where the new dam will tie into existing ground. With this length, a straight, fixed crest at the normal pool elevation of 899.6 ft-msl could pass approximately 13,500 cfs prior to the powerhouse/spillway structure being overtopped. This is less than half of the hydraulic capacity needed so a more hydraulically effective spillway discharge system will be needed at Lake Delhi Dam. Several spillway systems were reviewed for the alternatives analysis. A summary is provided in Table 4-7.

Table 4-7 New Spillway Option Comparison

Option	Suitable	Explanation
Fuse Plug	No	A fuse plug spillway consists of an earthen embankment overlaying a concrete spillway set several feet below the top of embankment. When the pool reaches the top embankment the earth is eroded away, exposing the concrete spillway. At Lake Delhi Dam, the concrete spillway could not be set low enough to provide sufficient hydraulic capacity.
Additional Lift Gates	No	Additional lift gates would require construction of a new section of spillway structure to essentially extend the existing spillway structure. However, bedrock drops away in this area so in addition to the additional cost of purchasing gates and hoisting equipment, the new concrete ogee spillway and operating platform would be founded on sand which would require expensive stability enhancements to make construction viable.
Pipes Through Embankment	No	In addition to concerns over seepage and maintenance, installing pipes through the dam embankment would not provide sufficient capacity and would require construction of a new intake and operating structure.
Labyrinth Weir	Yes	A labyrinth weir consists of a sharp-crested (vertical wall) in a zigzag pattern that allows a much longer crest length to fit within a shorter length of embankment. The longer crest length significantly increases the hydraulic capacity over a straight weir section. A labyrinth weir is a viable option for meeting hydraulic capacity requirements.
Pneumatic Crest Gates	Yes	A pneumatic gate system would consist of a concrete structure with crest control gates spanning the new spillway. They would consist of bottom mounted gate panels that could be lowered flush with the top of the new spillway. In their raised position they would be at or just above the normal pool elevation, but when lowered could provide an additional 5 to 10 feet of depth for discharging flood magnitude flows. Pneumatic gates are a viable option for meeting hydraulic capacity requirements.

From the initial review of spillway options, three spillway alternatives were developed for preliminary design and comparison. The three spillway alternatives are:

- **Dual Labyrinth Weir Spillway** – consisting of a lower principal labyrinth weir spillway set at the normal pool to discharge normal flows and a higher auxiliary labyrinth weir spillway set several feet above normal pool to discharge the required flood magnitude flows.
- **Single Labyrinth Weir Spillway** – consisting of a single labyrinth spillway set at normal pool to discharge normal flows but with sufficient hydraulic capacity to also discharge the required flood magnitude flows.
- **Pneumatic Gate Spillway** – consisting of a pneumatic gate system set at normal pool when raised to discharge normal flows and when lowered provides sufficient hydraulic capacity to discharge the required flood magnitude flows.

Exhibits showing plans and sections of the spillway alternatives are provided in Exhibits 5-7 Appendix F. All spillway alternatives consist of a concrete spillway slab and chute constructed over an earthen embankment with a concrete stilling basin at the end. All spillway alternatives were sized so with the three existing lift gates and the new spillway, the reconstructed dam could pass the ½ PMF without overtopping the powerhouse/spillway structure.

Dual Labyrinth Weir Spillway

Many dams have both a principal and auxiliary spillway. The principal spillway is designed for continuous use in passing normal flows and then the auxiliary spillway is designed for infrequent use in passing high magnitude flood flows. Because the auxiliary spillway is used infrequently, typically cheaper materials that are stable and safe for occasional but not frequent use can be used to construct portions of the spillway. Theoretically, this provides a cost savings in spillway construction. For the Dual Labyrinth Spillway option a principal labyrinth spillway would be used to discharge normal flows, used in tandem with the lift gates to discharge higher flows, and then the auxiliary spillway would engage at flood magnitude flows.

The Dual Labyrinth Spillway consists of a 120-foot long primary spillway labyrinth weir set at the normal pool elevation of 896.3 ft-msl and a 110-foot long auxiliary spillway labyrinth weir set at an elevation of 900 ft-msl.

The primary spillway discharges to a concrete chute with a concrete stilling basin at the toe. Training walls were kept straight for the preliminary design but could potentially converge slightly to save a small amount of concrete.

DNR design criteria require that at minimum the principal spillway be able to discharge the 50-year flood (~24,000 cfs) without engaging the auxiliary spillway. Combined with the spillway lift gates, the primary labyrinth weir spillway can discharge roughly the 100-year flood (~30,000 cfs). This would mean that the size of the principal labyrinth weir spillway could potentially be reduced so the combined gates and principal spillway discharge the 50-year flood and then the auxiliary spillway engages at flows exceeding the 50-year flood. However, it was determined during design that because the auxiliary spillway crest sits at a higher elevation than the principal spillway crest, the auxiliary spillway would have to be

upsized more than the principal could be downsized because the principal spillway can discharge more flow due to its lower crest. So the ½ PMF is controlling the design of both the principal spillway and auxiliary spillway.

The auxiliary spillway discharges to either a roller compacted concrete or articulated concrete block chute. These are cheaper surfacing than a concrete chute but are not meant to have continuous or frequent discharge over them. This is an additional reason for keeping a larger principal spillway because it would reduce the potential frequency of use of the auxiliary spillway. In the past three years a 50-year auxiliary spillway would have been used three times with the 2004, 2008, and 2010 floods whereas a 100-year auxiliary spillway would likely not have been used.

Concrete training walls will be provided between the principal and auxiliary spillways and on the southern edge of the auxiliary spillway to contain flow within the spillway chute.

Single Labyrinth Weir Spillway

With the ½ PMF being the controlling flood, the lower the weir crest elevation, the more flow that can be discharged prior to the upstream pool reaching the top of dam elevation of 906 ft-msl. Using a single labyrinth weir set at the normal pool elevation of 896.3 ft-msl allows a greater length of weir to be at the normal pool elevation, reducing the overall length of spillway required to discharge the ½ PMF.

The Single Labyrinth Spillway consists of a 180-foot long labyrinth weir set at the normal pool elevation of 896.3 ft-msl. The entire spillway uses a concrete chute and stilling basin.

For preliminary design the spillway crest was set at a single elevation. For normal operating conditions a better discharge scenario will likely be to provide a weir segment or series of notches a few inches lower than the rest of the weir crest. This will allow the discharge to be more concentrated rather than a thin film of water going over the entire crest and will help maintain the pool at a more constant elevation. This will be analyzed further and refined in final design. This adjustment will not impact the overall hydraulic capacity of the weir for passing flood flows.

Pneumatic Gate Spillway

Similar to the reasoning for developing the single labyrinth weir option, the pneumatic gates provide ½ PMF discharge capacity by essentially lowering the weir crest below the normal pool elevation during flood flows. Because the pneumatic gates can be lowered they provide an even greater flow depth for discharging floods over the spillway prior to the upstream pool reaching the top of dam.

The range of pneumatic gate settings was set to be from normal pool (896.3 ft-msl) down to 888.3 ft-msl which would be flush with the fixed concrete crest of the spillway. An electronic control system would regulate gate settings for normal flow, maintaining a constant pool elevation of 896.3 ft-msl. The length of the pneumatic gate spillway is 160 feet. Taller gates could reduce the length of spillway but also as the gates get taller the foundation gets larger and the downstream tailwater could impact discharge for floods approaching the ½ PMF magnitude.

Cost and Structural Considerations

Several factors were taken into consideration in the hydraulic design of the spillway alternatives. The ultimate controlling factor is passage of the ½ PMF design flood, but items impacting cost, structural stability and constructability were also evaluated.

The geometry of the labyrinth weir and pneumatic gate spillways were not just controlled by hydraulics but by structural issues as well. Labyrinth weir and gate heights were kept between 8 and 10 feet. A higher weir/gate height could provide more effective discharge, however when the wall or gate starts exceeding 10 feet, the additional structural and foundation requirements to make the overall structure stable start increasing to the point that making the spillway structure longer (i.e. more embankment length) is more cost-effective than trying to achieve a higher weir/gate.

A similar issue influences the steepness of the spillway chute. The steepness of the chute is controlled by the stability of the underlying earthen embankment. Hydraulically, a steeper chute could be used for the new spillway. However, the soil and stability parameters of the embankment and foundation control the steepness of the embankment.

Comparison of Three Spillway Alternatives

All three spillway alternatives have distinct advantages and disadvantages. Without considering cost or operating/maintenance requirements, the pneumatic gates appear to be the best option; they take up the least amount of area and provide normal pool control over a wider range of flows. However, pneumatic gates require additional mechanical and electrical systems that are not required for the labyrinth weir spillways. They also require additional operation and maintenance and have a service life of roughly 25 years, which is less than half of the service life of a concrete structure. With cost comparison between the single and dual labyrinth spillways, pneumatic gates would be more expensive to install and maintain.

The single labyrinth is 30 feet longer than the pneumatic gates but requires no operation. There is a greater sense of security knowing that the principal spillway is not subject to operation and maintenance of equipment. This is not to suggest that a labyrinth spillway will not require maintenance such as debris removal, but over normal day-to-day flows, the fixed labyrinth crest will provide a normal pool within 6 inches of 896.3 ft-msl for river flows up to 500 cfs without operating the lift gates.

With a shorter principal spillway, the hydraulic capacity for discharging flows within 6 inches of the normal pool is 300 cfs, so the lift gates would have to be used more frequently. The dual labyrinth weir is also 50 feet longer than the single labyrinth weir, so additional flow easement acquisition and grading will be needed along the south river bank to fit the dual spillways and chutes within the embankment and channel banks. The potential advantage of the dual labyrinth weir over the single labyrinth would be cost of construction where chute and stilling basin concrete (expensive) could be substituted for articulated concrete block or roller compacted concrete (cheaper) for the auxiliary spillway saving money on the overall construction cost. However after quantifying the additional cost of flow easement acquisition and grading and shaping the embankment and channel area for the larger dual labyrinth weir spillway the single labyrinth weir spillway is more cost effective than the dual labyrinth weir spillway.

The recommendation is to provide a single labyrinth weir spillway to discharge normal and flood flows at the dam. The single labyrinth provides less operation requirements, fits adequately within the channel area, will lower upstream flood elevations compared to the pre-breach dam, and effectively discharges the ½ PMF design flood.

Table 4-8 New Spillway Alternative Cost Comparison

Alternative	Cost
Dual Labyrinth Spillway	$2,805,000
Single Labyrinth Spillway	$2,267,000
Pneumatic Gate Spillway	$2,736,000

4.7 South Spillway Embankment (New)

The alternatives for the new (restored) earthen south embankment include a homogenous clay embankment, a zoned embankment (with a glacial till core and loess slopes), and a roller-compacted concrete (RCC) embankment. Long-term stability analysis and geotechnical recommendations indicate that there is little benefit in terms of stability or size of embankment footprint with a homogeneous embankment as compared to a zoned embankment. This is a result of the loess and till soils sampled in the vicinity of the project having similar composition and plasticity. RCC construction allows the use of steeper side slopes, reducing the size of the embankment footprint. However, the RCC option was not cost-effective due to the lack of on-site granular materials and the need to set up a mixing plant near the site. The recommendation is to construct a zoned embankment, utilizing both types of soils identified in the project vicinity. The central, core portion of the embankment will be constructed with lower-permeability glacial till. The core zone will be tied to the spillway structure seepage cutoff as well as the embankment underseepage cutoff.

Table 4-9 South Spillway Embankment Alternative Cost Comparison

Alternative	Cost
Homogeneous Clay	$1,080,000
Zoned Earth	$1,080,000
Roller Compacted Concrete (RCC)	$1,860,000

4.8 South Dam Embankment (Existing)

Two alternatives were evaluated for the existing south dam embankment, removal and replacement of the existing embankment material, or modification of existing embankment. The first alternative involves removing and replacing all existing in situ material and a portion of the seepage cutoff. Approximately 300 feet of existing embankment would be removed from the exposed face to the south abutment of the dam. The portion of the concrete wall and sheetpile cutoff system located above the embankment subgrade would also be removed to allow for construction of the new embankment. A new sheet pile cutoff would be driven and tied into the south side of the new spillway structure.

The other alternative for the existing south embankment is to tie the new spillway embankment into the existing embankment by "benching" into the existing embankment. For this alternative, the newly constructed spillway embankment section will be benched into the existing embankment with 8-foot horizontal by 2-foot vertical lifts. A small portion of the existing cutoff wall will be removed to allow for proper placement and compaction of new fill against the benches. The construction and integrity of the seepage cutoff within the existing embankment is not known. Therefore, a new sheet pile cutoff will be driven adjacent to the existing cutoff wall for the length of the existing embankment.

Removal and replacement of the existing embankment is expensive due to the large amount of material that will need to be removed and replaced. Considering the observed fair condition of the existing embankment and the substantially lower cost of the benching and modification alternative the recommended alternative is to bench the new embankment section into the existing south embankment with installation of a new sheet pile cutoff.

Table 4-10 South Dam Embankment Alternative Cost Comparison

Alternative	Cost
Remove and Replace Existing Embankment	$1,912,000
Bench Into Existing Embankment	$354,000

4.9 Minimum Flow Passage

One of the dam's operational requirements is to maintain a minimum flow to the downstream channel during times of very low river flow. Several alternatives were evaluated to maintain minimum flow passage:

- Refurbish or replace existing wicket gates for minimum flow control. (As was used previously).
- Install flow valves on new lift gates.
- Install valve or gate in new service spillway.
- Rehabilitate existing sluiceways at the base of the powerhouse structure.

To restore the operation of the dam to the pre-flood operational status and control minimum flow through the wicket gates would require considerable expense. The repairs would include replacing the old screw actuators on the wicket gates with a new system of hydraulic cylinders and a hydraulic power package. The wicket gates (originals from 1927) themselves would need to be refurbished to maximize the length of time until additional maintenance or replacement would be required. The HydroRake system, which was inundated in the flood, would need to be completely inspected - motors and electrical components replaced, equipment to be rewired, hydraulic oil drained, flushed, and replaced, and condition of the system bearings, belts, wheels, etc., would need to be determined.

Installation of flow valves near the bottom of two of the new lift gates is also an option. Compared to the wicket gates, this alternative will simplify the operation, minimize future

maintenance, and is a lower cost alternative. The dam's original lift gates are being replaced regardless of the selected alternative, so valves can be mounted on the new gates for minimal additional cost. The flow through the valves will also provide aeration of the water to help maintain downstream oxygen levels.

The third alternative is to install a minimum flow valve or sluice gate within the concrete weir on the new service spillway. This alternative offers similar advantages as the valves on the lift gate alternative in terms of maintenance, aeration, and low-cost. One disadvantage, however, may be access to the valve as the proposed spillway crest does not include an operator bridge. A separate platform would be required for this alternative. A cost was not developed for this option due to these complications, no benefit over the valved lift gate alternative, and added expense;

The final alternative to pass minimum flow is to pass the flow through the existing sluice pipes at the base of the spillway structure. Part of project construction will be to remove the riprap in the near upstream area of the spillway which should expose the sluice pipes. If located, the condition of the sluice pipes will be evaluated.

With installation of valves, the existing wicket gates and trash rake would not need to be repaired but they would still be available for eventual repair if hydroelectric generation is ever redeveloped. The valve and sluice pipe alternatives require significantly less future maintenance than the wicket gate option. There would be no hydraulic systems to maintain (oil, motors, hoses, controls, etc.) or belt conveyors.

The design alternative recommended for minimum flow passage is providing valves in the new lift gates since this option has the lower "known" cost component and would not have any of the dissolved oxygen issues from bottom discharge that the wicket gates and sluice pipes could have.

Table 4-11 Minimum Flow Passage Alternative Cost Comparison

Alternative	Cost
Refurbish Wicket Gates	$114,000
Valves in Lift Gates	$31,000

4.10 Fish Passage

The State of Iowa requires that fish passage be considered in design of any dam reconstruction. Lake Delhi Dam has an abandoned fish ladder on the south buttress wall. These types of steep concrete structures were typically installed in the 1920s–1940s and are not capable of passing native fish species.

The DNR was consulted on fish passage design at Lake Delhi Dam. The following design criteria for a fish passage system were provided:

- Constructed primarily of native stone materials.
- Sloped at a minimum of 20:1 (horizontal:vertical).
- Maintain a minimum wetted perimeter of 15 feet.

- Maintain a minimum cross-sectional depth of 1.5 feet.
- Incorporate resting structures and channel roughness.

A fish passage channel/structure was designed up the south river bank, crossing the dam on the south side of the new spillway structure. The channel would be a rock ramp/rapids configuration. The channel would be graded with a 5-foot wide bottom, 1.5H:1V side slopes, with a depth varying between 3 feet and 5 feet. Due to the 40-foot dam height, the 20H:1V nominally sloped channel is approximately 800 feet long and would feature approximately 20 rock-formed pools, ascending the riverbank and dam embankment. The channel entrance is approximately 500 feet downstream of the dam.

The rock channel would enter a flat, gate-controlled, open air, rectangular concrete channel at the top of the embankment. The concrete channel sides and bottom could be roughened or filled partially with rock. A sluice gate would be provided at the upstream end of the concrete channel. The flow line of the channel would be set slightly below normal pool to provide a constant flow through fish passage structure and channel. The sluice gate would be kept fully open during normal flow conditions but closed completely during times of high flows due to concerns over scouring and eroding the dam embankment area. Operation of the sluice gate could be manual, but it would be critical that the gate was closed during high flows. To improve the overall safety of the structure an automatic closure system is recommended and was included in concept design and costing.

Additional property will need to be purchased for installation of a fish passage channel.

A preliminary layout and section of the fish passage channel is provided in Exhibit 9 in Appendix F.

Installation of a fish passage channel is not recommended for the Lake Delhi Dam reconstruction for the following reasons:

- **Dam Safety:** The fish passage channel is essentially a small spillway. A gated closure at the upstream end would prevent high flows from scouring out the channel and portions of the dam embankment, but would depend on either automatic or manual closure which is subject to uncertainty.
- **Length of Channel:** Rock ramp/rapid fish passage installations have been successful on low-head dams throughout the Midwest. For the Midwest, Lake Delhi Dam is a relatively tall dam and there would be some uncertainty as to the potential usage of an 800-foot long fish passage channel because few have ever been installed in the region.
- **Invasive Fish:** In its reconstructed state, Lake Delhi Dam would provide an effective barrier to invasive fish such as Asian carp swimming upstream of the dam. A fish ladder would negate the dam acting as a barrier to unwanted invasive fish.
- **Cost:** The cost of the fish passage channel is significant, even relative to other structures being evaluated for the dam reconstruction.

- **Maintenance:** With most of the fish passage channel being constructed from loose, natural stone material, periodic maintenance of the rock pools would be required for the channel to remain effective from year to year. Debris and sediment would need to be removed from pools and rocks would need to be moved and replaced to maintain the channel.
- **Property requirements:** Installation of a fish passage channel will require purchase of additional property south of the dam. This has been incorporated into the alternative cost, but property acquisition could cause delays in starting construction.

Table 4-12 Fish Passage Alternative Cost

Alternative	Cost
Rock Ramp/Rapids Structure	$668,000

4.11 Recreational Amenities

Several recreational and public use amenities have been proposed as part of the dam reconstruction project. Proposed amenities include:

- Handicapped Accessible Fishing Pier.
- Canoe Portage Trail.
- Boat Ramp.
- Observation Deck.

The recommended option for providing a Handicapped Access Fishing Pier and Boat Ramp involve the construction of these amenities at a location separate from the dam. It is anticipated that land will be acquired and dedicated for these public access features. A separate location would enable better access to the lake within an area which is free from the inherent hazards of the dam and spillways.

An asphalt parking area is included at the south embankment area, where an adjacent observation deck will be constructed on the pool side. Preliminary alignments and provisions for a canoe portage across the south embankment were developed. Construction of this trail will require private property easements from landowners on the southwest and southeast quadrants of the dam site.

Since these added amenities are characterized as optional, the alternatives development as it pertains to each recreational amenity is simply whether to include or not include each amenity as part of the dam reconstruction project. The costs in the table below only include the construction costs of those features and do not include assumed property easement and acquisition costs. Those real estate costs are included as a separate item in the cost estimate.

Table 4-13 Recreational Amenity Costs

Amenity	Cost
Canoe Portage Trail	$59,000
Boat Ramp	$76,000
Observation Deck	$7,000
Handicapped Accessible Fishing Pier	$70,000

4.12 Sediment Control and Removal

As with any river, the Maquoketa River carries a sediment load. Sources of sediment are both watershed derived (exposed earth areas, farm fields, ditches, concentrated rural or urban stormwater with no sediment control, etc.) and river channel derived (bank sloughing and bed scouring). When river flow enters the Lake Delhi Pool, its velocity is significantly reduced and sediment is deposited in the upstream end of the lake. Over time, sediment deposition can raise the upstream lake bed to a point where it interferes with boating and recreational activities. Average sedimentation rates and volumes for Lake Delhi will be estimated during final design.

With the no pool, there is an opportunity to remove exposed sediment deposits at a lower cost than with traditional dredging methods used when the pool is up. For future sediment maintenance, a series of sediment control projects could be initiated in the Lake Delhi watershed to reduce the volume of watershed derived sediment that reaches the river with the goal of reducing the frequency of future dredging projects.

Section 5

Reconstruction Non-Alternative Features

5.1 Non-Alternative Features

Several Lake Delhi Dam reconstruction items did not warrant an alternatives analysis. These are major dam features that need repair or installation to return the dam to service but with no flexibility or options with how they are installed or restored so they were identified as Non-Alternative Features. Descriptions are provided in subsequent sections and costs have been included in the recommended project cost estimate which is provided in Appendix G.

5.2 Site Access and Utilities

Vehicular access for the powerhouse and existing spillway will be provided on the north embankment. The access area will be paved with asphalt concrete and will be secured with chain link fencing.

On the south embankment, the fish passage chute and gate control structure, if required, can be accessed from a new asphalt paved parking area, which will accommodate ten parking spaces. Steel beam guardrail will be utilized along the edges of the paved areas, and chain link fence provided to restrict access where necessary.

New storm drainage piping and catch basins will be utilized to drain the paved areas adjacent to the dam and existing gated spillway. Water service will be extended to the powerhouse area.

5.3 Powerhouse/Spillway Concrete Repair

The Structural Inspection/Evaluation performed by Stanley Consultants in September 2010 and subsequent site visits/inspections indicated that portions the powerhouse and spillway structures had experienced significant concrete deterioration, spalling and steel corrosion. Portions of the structures that were not visible, such as the lower upstream face of the powerhouse/spillway structure and stilling basin floor, will need to be inspected and evaluated for concrete repairs

during construction. All degraded concrete will require repair prior to returning the structures to service.

For the dam reconstruction, structural repair work was limited to what was necessary for the powerhouse and spillway to function as a water retaining structure and meet current dam design and safety criteria.

Repair work for the powerhouse and spillway includes:

1. Anchoring the existing powerhouse/spillway structure to underlying bedrock.
2. Waterproofing the roof slab of the powerhouse.
3. Removing and replacing deteriorated concrete on surface of ogee spillway sections with new concrete.
4. Removing and replacing deteriorated concrete on piers with new concrete.
5. Repairing/replacing gate slot concrete to accommodate the new gate system.

The bridge slab over existing ogee spillway was reconstructed in early 1990s. No significant deterioration was observed during inspections, and the bridge will no longer be open for general public access. No significant structural repair work was proposed for the spillway bridge but depending upon construction equipment/vehicle and load weights, access to the powerhouse portion of the bridge could be restricted.

The 2008 underwater inspection by J.F. Brennan Co. did not find any major structural issues with the lower upstream face of the dam and the stilling basin floor. These areas are assumed to have remained in good condition. The structures will be inspected once riprap is removed upstream and silt downstream to evaluate if repair is needed. At this stage, no significant item for repair of these areas was included in the cost estimate.

5.4 South Buttress Wall

The existing buttress wall is located on the south side of the gated spillway structure. No signs of instability were observed during the September 2010 inspection and subsequent site visits. Some localized areas showed signs of deteriorated surface concrete. The wall structure was determined to be suitable for rehabilitation to act as a transition between the gated spillway and the new spillway. Repair work for the south buttress wall includes:

1. Removal of the abandoned fish ladder.
2. Removal of the top portion of the upstream wall to allow free flow to the new spillway.
3. Raising portion of the downstream wall to accommodate the new spillway structure on the south side.
4. Repair of deteriorated concrete.

5.5 Electrical Service and Controls

Power Distribution

The existing 480/277-volt service to the dam is sized sufficiently for new mechanical and electrical equipment to be installed at the dam. Additionally, the two distribution panelboards HP and LP and the automatic transfer switch are all in good condition. These pieces of equipment can be removed and salvaged for reuse. The dry-type transformer should be inspected for any signs of water damage before a determination can be made if it should be reused. The main service disconnect switch and utility meter can be reused in their current location.

The remainder of the electrical distribution equipment, including the 208/120-volt distribution equipment, equipment disconnects switches, lighting and power circuits, conduit, wiring, motors, and other electrical devices, should be removed from the powerhouse.

The room on the main floor of the powerhouse currently housing the electrical equipment should be established as an electrical and control room. The room should be refinished to include a floor drain and equipment pads for any floor mounted equipment.

A new 480/277-volt, 200-ampere, 3-phase, 4-wire electrical service should be established in the refinished electrical room. A new service panelboard should be installed to include a main circuit breaker, so that the circuit breaker currently located in the stairwell of the powerhouse can be removed. The existing 208/120-volt panelboard and dry-type transformer can be reused.

Trash Rake and Hydroelectric Equipment

The conveyor equipment associated with the trash rake was underwater during the flooding and the motors, wiring, and control equipment associated with this equipment should be inspected and replaced prior to operating the trash rake. A new power feeder should be installed from the new electrical service to replace any existing wiring that may have been underwater.

The existing pool level monitoring and hydroelectric generator control equipment should all be removed as part of the dam restoration. The existing equipment has been extensively damaged due to the flooding and cannot be reused in its current state. The hydroelectric generators themselves can remain in place for possible use in the future with the addition of a control package, but nothing should be added to operate them at this time. If the wicket gates are repaired as part of the dam restoration, a new hydraulic power package and controls will be installed to operate them.

Lift Gates

As part of the dam restoration, three new lift gates will be installed in place of the existing gates. The dam operator previously purchased three electric motor driven screw actuators for operating the three gates. These three actuators can be modified and reused to operate the three new lift gates. The electric motors are 60 horsepower and rated for operation at 480-volts. The existing service would need to be increased in size to operate all three lift gates, however the existing electrical service to the dam will allow for operation of two lift gates

simultaneously. This is sufficient for the required operation of the dam. Three full voltage reversing starters should be purchased and installed in the new electrical room to operate the gates. Two of the new lift gates will be provided with a gate valve integral to the gate to provide for minimum flow.

A new control system should also be installed for operation of the lift gates and minimum flow gate valves. A single submersible level transducer should be installed on the pool side of the dam to monitor pool elevation. Automatic control of the gates can be derived based on existing pool elevation with manual control available at the control panel inside the electrical room.

A data connection should be installed to the dam in order to allow for remote monitoring of the dam and the possibility of remote control in the future. Additionally, the data connection can monitor data from the USGS monitoring station upstream of the dam. An autodialer will also be included with the control system to provide a telephone alarm if there is an issue with the automatic control system.

Emergency Generator System

While commercial power to the site is fairly reliable, it is recommended that an emergency generator be installed on site to operate the lift gates in case of power failure. There is no natural gas service to the site, so the new generator should operate on diesel fuel or propane. The installation of a diesel generator will be less expensive than a propane generator, due to the added cost for a propane tank and associated piping. The diesel generator can be provided with a sub-base diesel fuel tank integral to the generator and enclosure.

The generator should be sized at 125 kilowatts to allow for operation of two lift gates simultaneously. This will provide for a complete backup emergency system for the dam. The generator should be located on top of the powerhouse structure to allow for easy access and to be clear of any discharge path from the dam.

5.6 Safety Features

Safety is always a concern at a dam. In the upstream pool access (whether intentional or accidental) to gate intake areas, weir overflow areas, and spillway chutes need adequate warning and protection systems for all flow conditions. Downstream of the dam discharge and energy dissipation structures can cause rollers, eddies and vortices in the immediate downstream channel area that can be dangerous for recreational users of the downstream waterway. Appropriate warning signage and access control is also needed downstream of the dam.

The overall safety of the immediate upstream pool and downstream channel will be a major factor in the detailed design of dam reconstruction. Warning signage will be installed both upstream and downstream of the dam. The dam, spillway, and warning signage will be fully lit at all times. A buoy system, tie-off system, and boat restraining barrier will be installed upstream of the spillway. Warning lights and potentially sirens will be provided to indicate when lift gates are being operated. Fencing will be installed on the north embankment area and south embankment area to control access to the powerhouse and spillway structures.

Dam safety measures will be coordinated with DNR Dam Safety as design proceeds.

5.7 Archaeological Mitigation

A reconnaissance level archaeological survey was completed by Louis Berger Group (LBG) for the project site and upstream pool area. The survey identified 12 known archeological sites for supplemental investigation. In addition, LBG recommended that a supplemental, reconnaissance level survey be completed for portions of the pool area not studied as part of their original reconnaissance. Based on their findings, LBG recommended that the District enter into a Programmatic Agreement with the regulating state and federal agencies, detailing scope of additional investigations, so that permitting of the dam can proceed without delay.

If the investigations indicate sites are eligible for inclusion in the National Register, and/or burial sites are identified, mitigation of potential adverse effects of the project construction and restoration of the original pool level will need to be mitigated. At this time, it is not known if or what mitigation may be required, so a budgetary amount has been included in the recommended project cost estimate.

5.8 Property/Easement Acquisition

The District will need to acquire temporary construction access easements, perpetual easements, and/or property for the construction and future maintenance and operation of the repaired Lake Delhi Dam. Iowa DNR Technical Bulletin 16, "Design Criteria and Guidelines for Iowa Dams," outlines property ownership and easement requirements for dams. The bulletin states that "The determination of lands, easements, and rights-of-way required for the construction, operation and maintenance of a dam project are considered part of the design process."

Temporary easements are required to allow access for construction of the project. These areas will be restored, if damaged, and easement canceled upon completion of construction. Perpetual easement or ownership is required for areas occupied by the dam structures. This is to ensure the District is allowed access to all areas of the project for inspection and maintenance. Perpetual easement or Ownership also places decision authority with the District regarding any construction or modification to these parcels. Perpetual easement or ownership is also required for areas that may be inundated during all flows, up to and including the design flood event. This is to ensure that no construction on, or modifications to any parcels required for safely passing predicted flows are completed without District approval.

The property research completed by Gibbs Engineering and Survey, combined with the preliminary design completed for this study have indentified approximate limits of temporary, and perpetual easement/ownership requirements for the proposed project. As the project moves to final design, design refinements and legal boundary surveys will determine the required easement and ownership boundaries. A budgetary amount was included in the recommended project cost estimate for property/easement acquisition.

Section 6

Construction Sequencing

6.1 Construction Sequencing

Construction of the Lake Delhi Dam Reconstruction Project will be broken into two phases. The two phases of work may be let as a single project or bid under separate construction contracts. Phase 1 will involve work at the existing powerhouse/spillway and north embankment. The Phase 1 work tasks include:

- Powerhouse/spillway stabilization (rock anchors).
- Powerhouse/spillway concrete rehabilitation.
- Stilling basin silt removal.
- Downstream abutment wall reconstruction.
- Electrical system upgrades.
- Lift gate and hoisting equipment replacement.
- Bridge deck (powerhouse roof) improvements.
- North embankment walls demolition and reconstruction.
- Upstream riprap removal.

It is anticipated that the Phase 1 work would take place during the spring and early summer months of the construction season. Water would continue to flow through the current river channel during Phase 1 construction and an upstream pool would not be maintained. Upstream and downstream cofferdams, along with a dewatering operation would be required to maintain workable site conditions.

Phase 2 will involve all work south of the existing powerhouse/spillway. The Phase 2 work tasks include:

- Buttress wall rehabilitation.
- South embankment construction.
- Seepage cutoff installation.
- New spillway construction.
- New stilling basin construction.
- Channel grading.
- Scour protection.
- Public amenities.

It is anticipated that the Phase 2 work would take place during the late summer and fall months of the construction season, with some site finishing and cleanup performed the following spring. During Phase 2 construction, water would be diverted through the existing gated spillway, requiring construction of a substantial upstream cofferdam to raise an upstream pool and establish flow through the gated spillway. Note that existing sluice pipes at the base of the powerhouse will be investigated during Phase 1 construction. If the sluice pipes are operational, there is the potential to divert flow through the sluice pipes during Phase 2 construction, reducing the size of the upstream pool and substantially reducing cofferdam and dewatering construction costs. Under either scenario, upstream and downstream cofferdams, along with a dewatering operation would be required to maintain workable site conditions during Phase 2 construction.

6.2 Construction Staging

Given the limited amount of space and right-of-way at the dam site, alternate construction staging areas will be required. Potential Phase 1 and Phase 2 construction staging areas are located within close proximity to the site. The Owner may elect to designate specific staging areas for construction, or allow the construction contractor to make arrangements with landowners for construction staging.

6.3 Cofferdams and Dewatering

As discussed in Section 6.1, upstream and downstream cofferdams and dewatering operations will be required for both phases of construction. Cofferdam design and dewatering design will be the responsibility of the construction contractor. For cost-estimating purposes, trapezoidal earthen cofferdams were assumed to be constructed to one foot above the five-year return period flood event for Phase 1 construction and one foot above the two-year return period flood event for Phase 2 construction. See Exhibits 10–11 in Appendix F for conceptual cofferdam layouts and cross sections. A deep-well dewatering system was also assumed for each phase of construction for cost-estimating purposes.

Cofferdam construction may require temporary construction right-of-way easements from adjacent property owners.

6.4 Borrow Material

The construction contractor will be responsible for obtaining borrow material from a private source for embankment construction. Soil borings have been advanced at potential borrow site areas to verify that proper embankment construction materials are available in the area. The upper material (loess) may require farming (drying) prior to placement. For cost estimating purposes, a five-mile round-trip cycle was assumed for material hauling.

6.5 Riprap

It is assumed that a sufficient amount of riprap is available on site for the scour protection requirements of the project. The construction contractor will be responsible for excavating, stockpiling, and placing on-site riprap between the two construction phases. If additional riprap is required, it will likely be imported from a neighboring quarry.

Section 7

Cost Estimate and Construction Schedule

7.1 Cost Estimate

A preliminary cost estimate was developed for alternative concepts to assist with evaluation and selection. Recommended alternatives were then incorporated into a preliminary project estimate, representing the current estimated cost of the repair project. Unit costs for project features were taken from RSMeans and the Stanley Consultants database of recently completed dam construction projects. All costs from previous projects were adjusted for location and inflation.

Several cost items were developed based on visual/surficial inspections, document review, and assumptions of typical conditions. During construction, there is always the possibility that unknown issues or conditions will be encountered, impacting the cost of the project. In addition, this estimate represents a preliminary stage of design. As design progresses, the construction cost estimate will be refined. A 20% contingency has been added to the preliminary cost estimate to account for unknowns and future design development.

Estimates of engineering design fee (assuming 7% of construction cost) and engineering services during construction (assuming 36 weeks of construction) have been included in the recommended project. The total estimated cost was then escalated 5% to account for construction next year. The total estimated preliminary cost for the recommended project is $11,870,000. Both the recommended project cost estimate and the individual cost estimates for comparing the reconstruction alternatives are provided in Appendix G.

7.2 Schedule

A preliminary construction schedule was created using Primavera P7 software, and is presented in bar chart format using the critical path method of scheduling. The two-phase approach from Section 6 was used, with Phase 1 awarded April 1 and construction starting mid-April. Phase 2 is awarded in late June, with construction starting in mid-July.

A preliminary schedule was created for two different scenarios. The first schedule was developed using a five-day workweek calendar with weather and holidays. The second schedule was created using the same construction activity durations, but uses a six-day workweek calendar with weather and holidays. With the six-day workweek calendar, critical construction activities are completed before severe winter weather sets in. In the five-day workweek scenario, the schedule is extended into the winter months, and further extended by severe weather allowances in the calendar. The six-day workweek calendar schedule is provided in Appendix G.